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Proceedings of the American Society of Civil Engineers

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NEW TEST FOR CONTROL OF COHESIVE SOILS IN ROLLED-FILL

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(Proc. Paper 933)

SYNOPSIS

A new test, called the "drop" test, has been devised by the writer to overcome the difficulties entailed in the effective control of the placing of rolled-fill under modern rapid rates of construction.

A small sample of the soil from the borrow pit, or as delivered to the work site, is compacted into a specimen ring four inches in diameter and one and one-half inches high.

The specimen of soil prepared in this manner is weighed and, after removal from the ring, is dropped on edge from a fixed elevation onto a concrete slab, and its reduced height above the flattened surface is measured.

With the wet density and the reduced height as the only known characteristics of the soil; it is possible, by means of the chart supplied, to determine the moisture content of the soil relative to its optimum moisture content under the form of compaction employed and its optimum wet density.

In a few minutes it is practicable to determine the answers to the important questions as to the suitability of the moisture content of the soil for rolling, any adjustment in the moisture content that may be necessary, and the desired density to be attained by the rolling procedure.

The results of tests on thirteen samples of soil are shown and demonstrate the effectiveness of this method of testing where the Standard AASHO optimum moisture content varies between 12 and 30 per cent.

The drop test is shown to represent a fundamental property of the moist soil. The results can be applied to all cohesive soils under any chosen standard of compactive effort.

INTRODUCTION

When the construction of the rolled-fill section of the Big Eildon embankment commenced, considerable difficulty was experienced in controlling the

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optimum moisture content and the rolled density of the fill. The range of moisture permitted was from two per cent on the dry side to one per cent on the wet side of optimum. The speed of placement was such that about thirty minutes were available for the determination of the moisture content of the soil in the borrow pit, also as spread on the embankment; and how much water, if any, should be added before commencing rolling. After 12 passes of the sheepsfoot roller the density and moisture content had to be determined within a similar time limit in order to check the density of the fill.

The determination of the moisture content was made within 20 minutes drying time by the use of infra-red ovens. This was not accurate and still did not overcome the time difficulty. In addition, as pertains to several Victorian dam sites, despite extensive investigations before the commencement of the work of constructing the embankment; the soils were found to vary so much that the compaction characteristics of the soil in use at any particular moment were not known.

After a visit to the site, a method of approach to the solution of the problem was proposed and, after investigation, the present drop test procedure was evolved.

The drop test determines a fundamental property of each soil, so that if a drop test is made on a soil having a Standard AASHO optimum moisture content of more than approximately 12 per cent, within the range of -1.8 to +3.0 per cent of optimum, its moisture content relative to an unknown optimum can be determined from the result of one test.

The actual moisture content, the actual optimum moisture content, and the actual optimum dry density of the soil are all unknown; yet it can be rolled at its optimum moisture content to its optimum dry density as a result of the supply of the requisite information to the field staff by means of the drop test technique.

Test Procedure

A representative sample of the soil at its natural moisture content is quickly broken up and compacted into a 4-in. diameter by 1 1/2-in. height specimen ring retained in a compaction cylinder (Figure 1).

A portion of wet soil weighing 1.65 lb., with the inclusion of all pebbles up to 1/2-in. maximum diameter, is compacted under the equivalent of 25 blows of a 5.5-lb. tamper falling through a height of 18 in. to form 2 in. of compacted soil; that is, 19 blows on this weight of wet soil. The equivalent of any other desired compactive effort can be used, Modified AASHO compaction requires 47 blows of a 10-lb. tamper falling through a height of 18 in. on the same weight of wet soil.

During the compaction the base of the compaction cylinder is bedded on the ground, or on a sand bed in the laboratory, and held firmly in contact therewith.

The upper face of the compacted soil is trimmed true to the ring dimension by means of a hardened steel straight-edge, and both faces are protected against evaporation losses by methyl methacrylate resin plates or tin foil. The weight of the specimen is determined to 0.1 gram, and the wet density in pounds per cubic foot recorded.

When an appreciable quantity of particles coarser than 2 mm. in diameter is present, it is desirable to test the whole specimen as compacted without attempting to trim it to size. When this is necessary, the soil is compacted

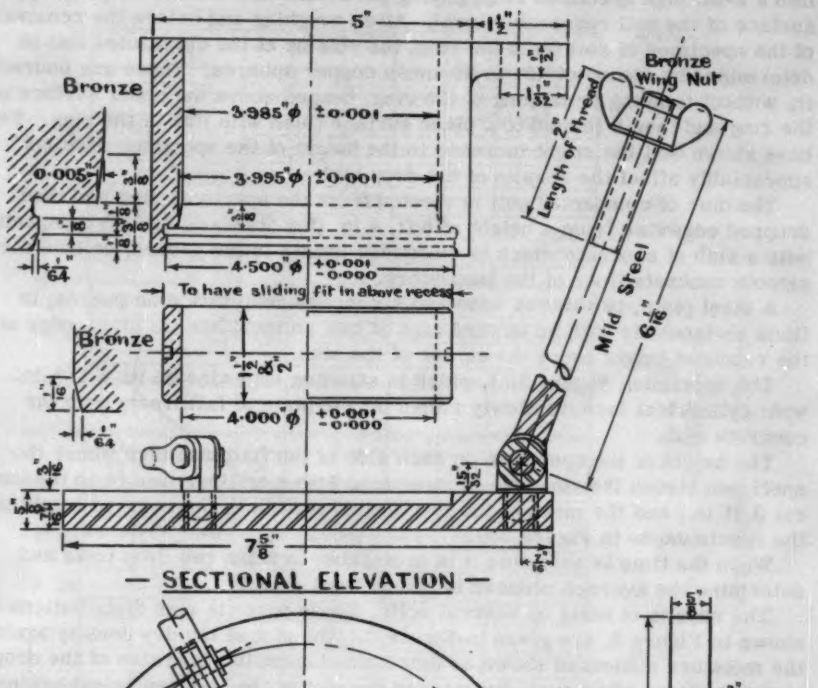

FIG. 1.–COMPACTION CYLINDER, BASE AND SPECIMEN RING

into a 2-in. high specimen ring, paying particular attention to keep the upper surface of the soil reasonably level. After weighing and before the removal of the specimen of soil from the ring, the volume of the compacted soil is determined by means of 40- to 60-mesh copper spheres. These are poured in, without tamping or tapping of the ring, heaped above the upper surface of the ring and then trimmed to a plane surface flush with that of the ring. Tests have shown that the slight increase in the height of the specimen does not appreciably affect the results of the drop test.

The disc of compacted soil is ejected from the specimen ring and then dropped edgewise from a height of 6 ft. 4 in. (for Standard AASHO compaction) onto a slab of concrete which is embedded firmly in the ground, or onto the smooth concrete floor of the laboratory.

A steel plate, two inches wide and six inches deep with side guards, is fixed horizontally with an upward rise of half an inch, and its lower edge at the required height above the center of the slab.

The specimen, Figure 3(h), which is standing edgewise on its 1 1/2-in. wide cylindrical face, is slowly rolled off the plate to fall freely onto the concrete slab.

The height of the specimen on each side of the flattened face where the specimen struck the concrete is measured with a calliper square to the nearest 0.01 in., and the mean of the two measurements is the reduced height of the specimen, h in Figure 3(i).

When the time is available it is preferable to make two drop tests and determine the average reduced height and wet density.

The results of tests on several soils, whose particle size distributions are shown in Figure 2, are given in Figure 3. The plot of the dry density against the moisture content is shown as determined from the densities of the drop test specimens after oven-drying, and the plot of the reduced height against the moisture content is also shown for each sample of soil.

The soils were compacted under the equivalent of Standard AASHO compaction. The soil corresponding to d' of Figure 2 was tested, and gave similar results to those for soil d at a slightly lower optimum moisture content and higher optimum dry density.

It is clear that the lower intersection point on the drop test curve coincides with the optimum moisture content of the soil.

The compaction procedure does not exactly duplicate that obtained by the writer's standard procedure[1] using Standard AASHO compactive effort, but is a very close approximation to it, see curve c' in Figure 3 for standard compaction on soil c.

While the Standard AASHO compaction uses varying quantities of soil to make a fixed layer thickness, it is considered that the present practice of the use of a uniform weight of wet soil for all compactions is a step in the right direction. Such uniformity in the weight of soil taken is an essential feature of the drop test procedure.

When investigating the procedure in the laboratory all the soils are tempered separately for each moisture content after mixing, in accordance with the writer's practice, for 1-3 days before testing.

1. "Handbook of Methods of Testing Soils," by J. MacNeil Turnbull, Government Printer, Melbourne, 1945, p. 32. (Distributed by Tait Book Co. Pty. Ltd., Melbourne.)

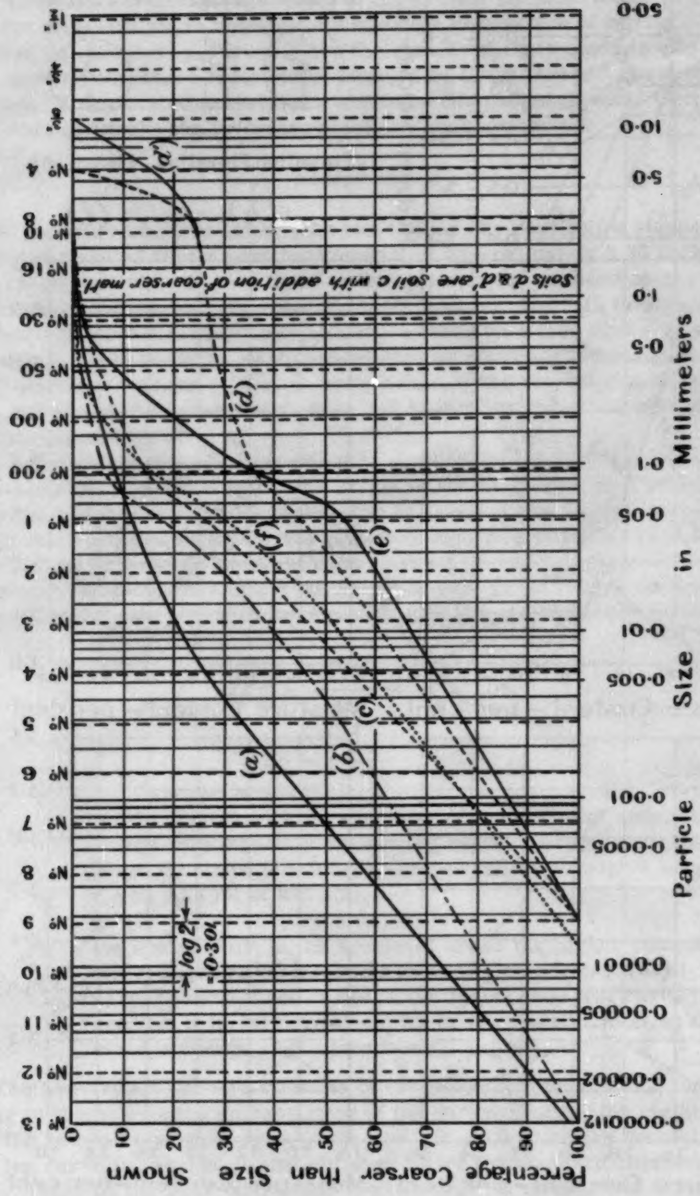

FIG. 2.- PARTICLE SIZE DISTRIBUTION CURVES.

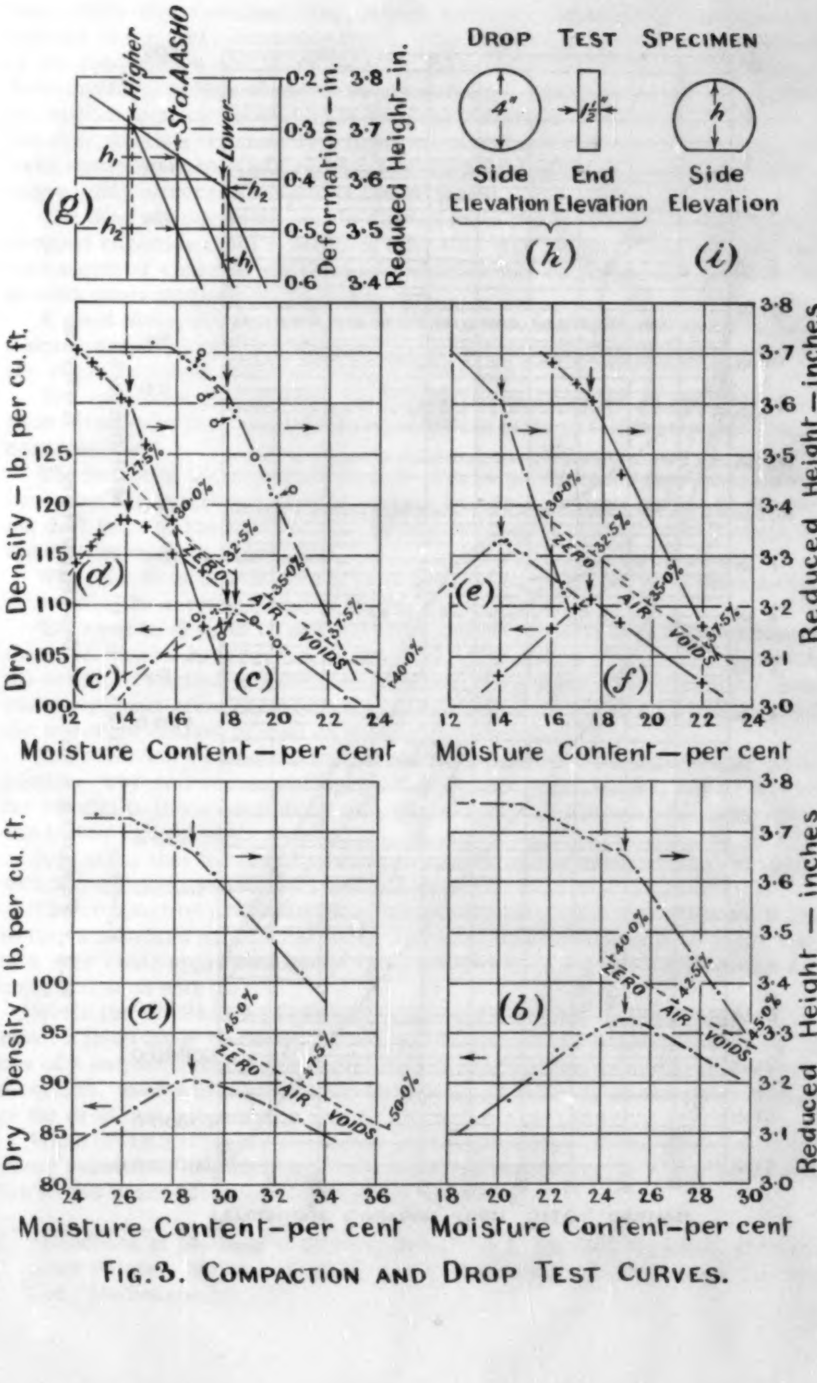

FIG. 3. COMPACTION AND DROP TEST CURVES.

In the field this tempering is not required as the natural moisture content will have ensured complete wetting of the crystal lattice. Water can be added and the drop test made immediately after the completion of mixing. The small circles on curve c of Figure 3 are the results of tests made in this manner and show no departure from those obtained by the standard laboratory procedure. If desiccated desert soils are to be compacted it would then be preferable to make the laboratory tests without tempering.

Interpretation

At moisture contents lower than two to four per cent below standard optimum the reduced height remains constant at approximately 3.70 inches. As the moisture content is increased to the optimum the reduced height decreases slowly at a uniform rate by approximately 0.10 in., and with further additions of water it decreases uniformly at a more rapid rate (see curve c of Figure 3). This curve is an individual characteristic of the moist soil, and for a particular height of drop is independent of the dry density of the compacted soil. In this respect the drop test is similar to the Proctor penetration resistance test.

When a heavier compactive effort is desired than that obtainable by the equivalent of Standard AASHO, the height of the drop must be increased otherwise the lower intersection point on the drop test curve will remain at the same moisture content as for the lighter compactive effort. For the Modified AASHO compaction the height of drop required is 110 inches, and for other standards of compactive effort the height of drop is increased or decreased from 76 inches according to the equation (see Figure 3(g)):

$$H = \frac{76}{h_1} \times h_2 \quad \text{in.}$$

where H = the required height of drop,

h_1 = the deformation of the specimen at the optimum under the lower compactive effort on the wet leg curve, extrapolated if necessary, under 19 blows of a 5.5-lb. tamper falling through a height of 18 in. on 1.65 lb. of wet soil,

= (4 - h) in. (Figure 3 (h), (i))

h_2 = the deformation of the specimen under the higher compactive effort at the same moisture as for h_1 on a line parallel, if necessary, to the wet leg curve of the 76-in. drop test curve to intercept the original dry leg curve, or its extrapolation, at the desired optimum moisture content.

The new drop test curve consists of two straight lines, that on the dry leg being identical with the extrapolation of the curve obtained for the Standard AASHO compactive effort down to the new optimum moisture content. The wet leg curve is parallel to that obtained under the standard compactive effort. Tests on the same soil under the two limiting compactive efforts and the same height of drop of 76 in. result in identical curves. The point of intersection is at the higher optimum moisture content and the wet and dry legs coincide, except that the test results under the heavier compactive effort on

the dry side of optimum extend to a greater reduced height than is possible under the lighter compaction. Use can be made of the latter characteristic by compacting another specimen of the drier soil under a higher compactive effort, but retaining the height of drop pertaining to the standard of compaction in use. The wet density of the specimen compacted in the standard manner would, of course, be used with the new reduced height.

For a 40-blow compaction, in place of the Standard AASHO compaction using 25 blows, the height of drop is 84 inches.

The vertical cracks that form on the dry side of optimum supply a useful guide for any particular type of soil, in extreme cases the specimen is shattered.

Bulging at the sides, when due to internal collapse and not the plasticity of the soil, indicates a false reduced height that is lower than normal. This type of failure is likely with poorly graded soils containing large quantities of silt. The drop test curves for such soils follow the standard pattern, and by making a series of drop tests at varying moisture contents it is possible to locate the optimum correctly.

Applications

The results of a series of tests on thirteen soils varying in optimum moisture content between 12 and 30 per cent are shown in Figure 4. In this chart the wet density as determined from the drop test specimens has been plotted against the reduced height after a drop test. These points are seen to lie on curves consisting of three straight lines (dashed in the Figure). The points for each soil at intervals of 0.2 per cent of moisture above and below optimum are shown as obtained from the test curves.

Superimposed are a series of curves, indicating the position of any test result relative to the optimum moisture content of that particular soil, at intervals of 0.2 per cent of moisture between the limits of 2.0 per cent below to 3.0 per cent above optimum. The dashed lines for each soil between the limits of 0-1 and 1-4 per cent wetter than optimum are parallel to OP and PQ, respectively. The origins of the other lines are shown on the Figure.

This is a universal chart and having made one drop test, if the natural moisture content of the soil falls within the range of the chart, the departure of its actual moisture content from optimum and its optimum wet density are known immediately.

For example, the point x in Figure 4, corresponding to a soil having a wet density of 127.0 lb. per cu. ft. and a reduced height of 3.68 in., shows that the soil moisture is 1.4 per cent below its optimum at a wet density of 131.5 lb. per cu. ft. and a reduced height of 3.615 inches, indicated by the point z. If this moisture content is within the desired range then rolling is commenced and continued until the wet density of the soil is 131.5 lb. per cu. ft., or such value as is specified.

A specimen cutter, 0.0005 in. smaller in diameter than the specimen ring, is used to take an undisturbed sample of the rolled soil. This sample is transferred to the specimen ring for the determination of its wet density and the reduced height after a drop test. Confirmation, or otherwise, of satisfactory rolling is available immediately.

When the soil contains an appreciable proportion of particles coarser than 2 mm., the wet density of the fill is determined by one of the usual methods, and the soil that has been removed from the hole is recompacted into the

(Lines A,B,C & D radiate from point 3·655 in. x 104·0 lb. per cu. ft.
Lines D, E, F & G radiate from point 3·725 in. x 86·7 lb. per cu. ft.
Lines between D & E are evenly spaced along the line joining points 3·655 in. x 104·0 lb. per cu. ft. and 3·725 in. x 86·7 lb. per cu. ft.
Lines H - M radiate from point 2·29 in. x 176·5 lb. per cu. ft.
Lines M - N are parallel.)

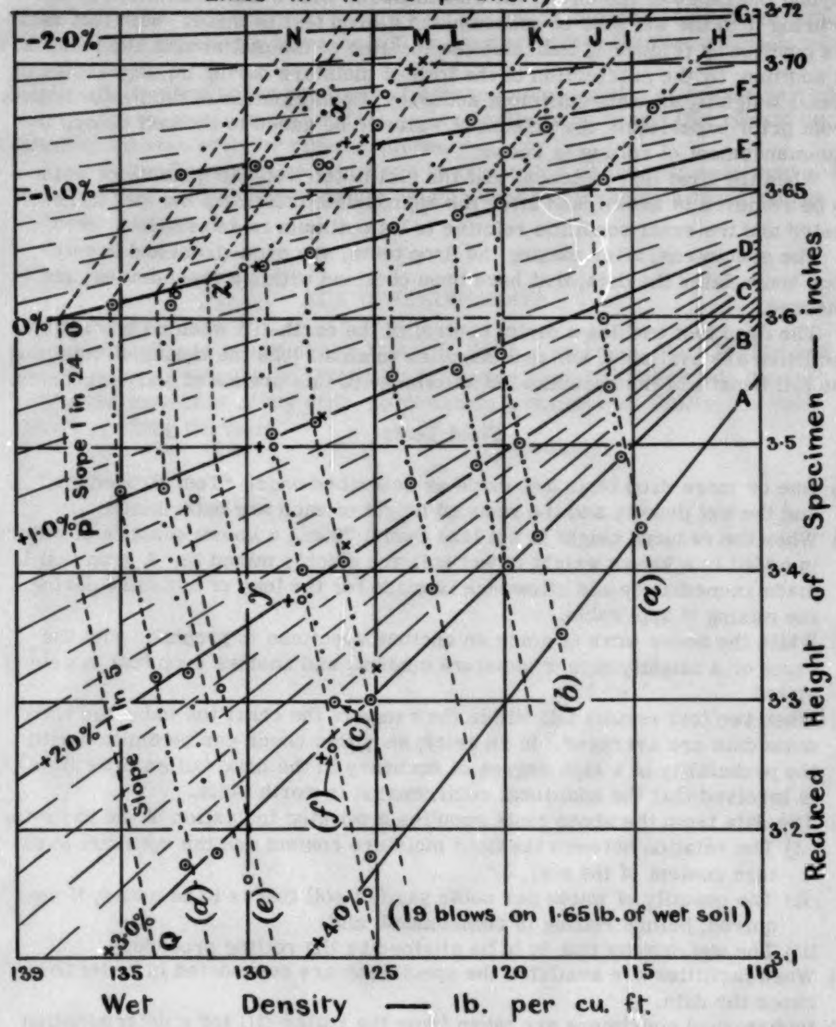

FIG. 4. DETERMINATION OF MOISTURE CONTENT OF A SPECIMEN OF SOIL RELATIVE TO ITS OPTIMUM MOISTURE CONTENT.

specimen ring for a drop test determination of its relative moisture content. If the soil is being rolled on the dry side of optimum, water is added to the drop test sample as described later in order to avoid errors due to the increased density of recompacted soil near or on the dry side of optimum, or the field density is used on the chart.

When the first test indicates that the soil is on the dry side of optimum and beyond the range of the chart, the soil is mixed with a known addition of water to bring it to the wet side of optimum and a drop test is made. With this data, the position y (Figure 4) indicates the location of the optimum at the point z. In addition, by the subtraction of the loss of moisture during mixing; either by actual weighing or, with sufficient accuracy, by estimation of the probable loss from prior experience; the amount of water to be added to the soil before the commencement of rolling is known.

When the first test indicates that the soil is too wet, the quantity of water to be removed is known, and after the appropriate treatment the soil is retested and the exact condition relative to its optimum is determined.

The specimens, after making the drop tests, are oven-dried and twenty-four hours later the data, that have been obtained within a few minutes, are checked.

The drop test enables a strict control of the earth-fill when no laboratory facilities are available, and thus supplies to small jobs the means of obtaining the full benefit of the accumulated knowledge in this branch of soil mechanics.

Field Tests

1) One or more drop tests are made as described under "Test Procedure," and the wet density and the reduced height of each are determined.
2) When the reduced height is not less than 3.70 in., a known quantity of water is added to a known weight of wet soil and quickly mixed in. A drop test is made immediately and allowance is made for the loss of moisture during the mixing if applicable.
3) While the above work is going on another specimen is prepared with the same or a slightly higher moisture content, and another drop test is completed.
4) When two test results fall within the range of the chart the indicated optimum data are averaged. In an emergency this check can be omitted with the probability of a high degree of accuracy in the data, but so little time is involved that the additional confirmation is worth while.
5) The data from the above tests supplies a working indication of the following:
   i) The relation between the field moisture content and the optimum moisture content of the soil,
   ii) The quantity of water per cubic yard of soil that is to be added, if required, before rolling is commenced, and
   iii) The wet density that is to be attained by the rolling procedure.
6) When facilities are available the specimens are oven-dried in order to check the data.
7) Undisturbed specimens are taken from the rolled-fill for a determination of the final density and, from a drop test, the moisture content in relation to optimum.

CONCLUSION

A new method of testing a soil for a quick determination of its wet density and its moisture content relative to its unknown optimum is described. The results of tests on a wide range of soils are supplied and a chart to permit direct application to most soils is included. The test has been tried out in the field and found to be the only method capable of supplying the desired data within the short time available with the use of modern rapid construction procedures, especially when the soil is near or wetter than its optimum moisture content.

Its adaptability to the presence of a reasonable quantity of coarse particles (up to 1/2-in. maximum diameter) enables soils to be tested that cannot be handled satisfactorily by other methods.

The desired compactive effort can be duplicated on the test specimen and the drop test from the appropriate height supplies all the data required for efficient control.

ACKNOWLEDGMENTS

Appreciation is expressed to Lewis R. East, Chairman, State Rivers and Water Supply Commission, Victoria, for permission to publish this paper; and to the members of my staff, particularly Lawrence A. Reilly, for their care in making the tests.

CONCLUSIONS

[illegible]

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ACKNOWLEDGMENT

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REFERENCES

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STABILIZATION OF MATERIALS BY COMPACTION

W. J. Turnbull,[1] M. ASCE, and Charles R. Foster,[2] A.M. ASCE
(Proc. Paper 934)

SUMMARY

The desired stabilization of a soil can often be achieved by compaction at the proper water content. This paper shows, for cohesive soils, how strength varies with both water content and density and how variations in rolling and lift thickness affect the density and strength obtained in field compaction.

INTRODUCTION

The term "soil stabilization" is usually defined as the process by which soil structures of the desired bearing capacities are produced. Most definitions carry the thought one step forward by stating that the desired capacities are produced by the addition of other substances. Soil stabilization processes that include the addition of other substances require processing and mixing of the soil and, in most cases, adjusting the mixture to a desired water content, followed by compaction. In many instances the desired bearing capacity can be obtained by the latter two processes, water adjustment and compaction, without adding another substance; therefore, it seems logical to consider compaction as a means of soil stabilization.

The attainment of a high degree of strength is usually associated with a high degree of compaction. This simple association is not entirely correct as the attainment of a high degree of strength, or for that matter any desired degree of strength, in cohesive soils is contingent upon the presence of the proper water content and density, and both of these must be considered in all cases. It is the purpose of this paper to show, for cohesive soils, (a) how the strength varies with both water content and density, and (b) how variations in rolling and lift thickness affect the density and strength obtained in field

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compaction. Since in some cases a controlled rather than a high strength is desired, data are presented for a wide range of conditions.

ACKNOWLEDGMENT

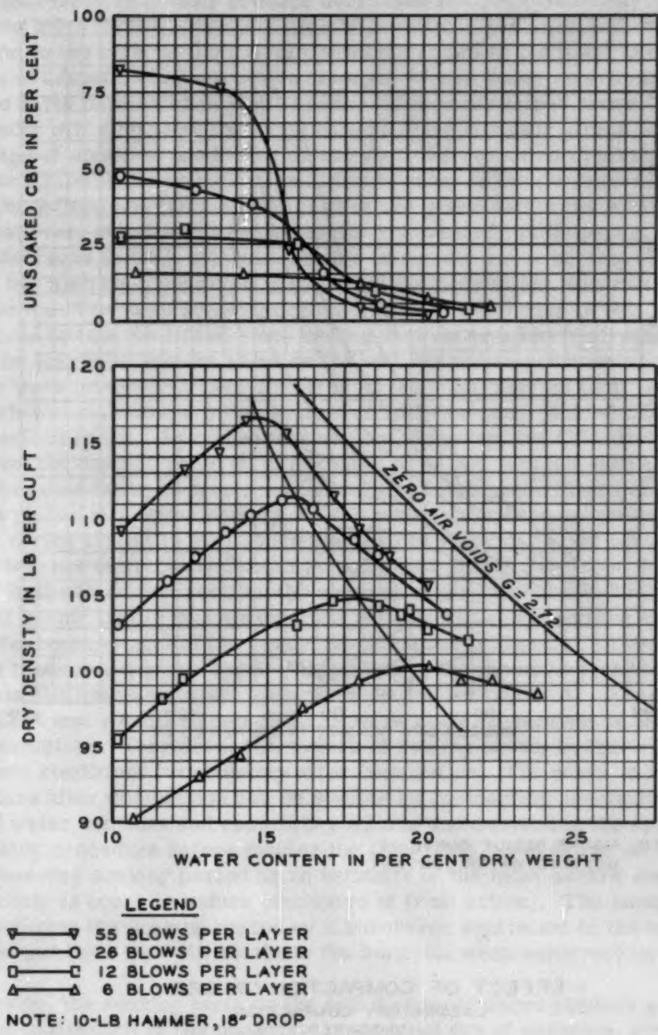
The studies discussed herein have been conducted under the direction of the Office, Chief of Engineers, as a part of the over-all development of design criteria for airfield pavements being accomplished for the U. S. Air Force. Messrs. Gayle McFadden, T. B. Pringle, and F. B. Hennion of the Airfields Branch of the Office, Chief of Engineers, monitored the study. Guidance for this study has been furnished by the following consultants: Professors A. Casagrande, D. W. Taylor, and K. B. Woods, Mr. O. J. Porter, and Dr. P. C. Rutledge. The laboratory and field work was accomplished by the Corps of Engineers' Flexible Pavement Laboratory, Waterways Experiment Station, Vicksburg, Mississippi. The results have been published by the Waterways Experiment Station in a series of reports entitled "Soil Compaction Investigation," Technical Memorandum 3-271. Six reports of this series have been published and two are in preparation.

Variation of Strength with Moisture and Density

The necessity to consider both water content and density in studying the strength of compacted soils is well illustrated by the laboratory CBR test. The upper part of fig. 1 shows the water content-CBR relationships and the lower part shows the water content-density relationships that are developed when a group of samples of a cohesive soil are compacted at a range of water contents and compactive efforts and subsequently tested for CBR value. The procedure of using more than one compactive effort yields a range of densities for each water content. The results shown on fig. 1 are for a lean clay which is classified according to the Department of Army Unified Soil Classification System as ML-CL. The liquid limit is about 38 and the plasticity index about 13. The soil was obtained from the zone of weathered loess native to the grounds of the Waterways Experiment Station at Vicksburg, Mississippi. It will be noted in the lower part of fig. 1 that for any one compactive effort, the density increases as the water content increases until an optimum water content is reached beyond which the density decreases with increasing water content. Also, as the compactive effort increases (additional blows), the maximum density increases and the optimum water content decreases. Generally, the optimum water contents can be connected with a smooth curve which has approximately the same shape as the zero air voids curve.

It will also be noted in the upper part of fig. 1 that as the water content increases, the CBR generally decreases. The curves in the upper part of fig. 1 were developed from CBR tests performed on the samples compacted for the moisture-density relationships. Each point in the upper part of fig. 1 has a corresponding point in the lower part.

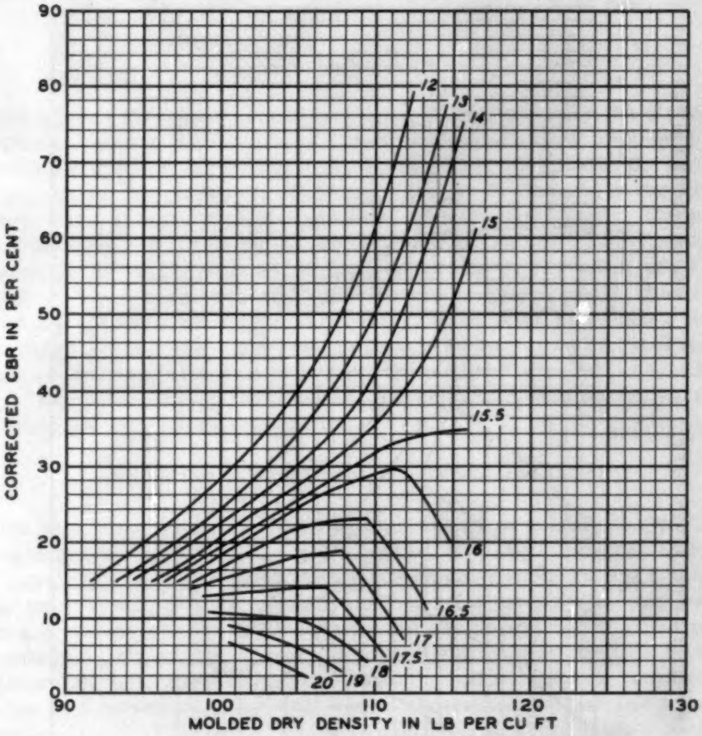
The moisture-density-CBR relationships shown on fig. 1 can be plotted to show the relationship of CBR versus density for equal water contents as shown on fig. 2. The plot on fig. 2 was made by reading, at given water contents, density values and corresponding CBR values from both the upper and lower plots of fig. 1. These values were plotted on fig. 2 and joined by smooth curves. It will be noted that for the lower water contents, the CBR increases



MOLDING WATER CONTENT VS DENSITY AND CBR

LABORATORY DYNAMIC COMPACTION

FIGURE 1



NOTE FIGURE BESIDE CURVE IS MOLDING WATER CONTENT.

EFFECT OF COMPACTION ON CBR
LABORATORY COMPACTION
UNSOAKED

FIGURE 2

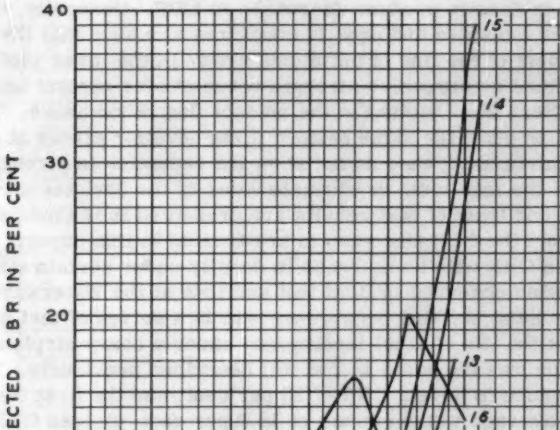
with an increase in density. At the intermediate water contents, the CBR increases with increases in density up to a certain value of density and then further increases in density produce decreases in CBR. Generally, the CBR decreases when the moisture and density conditions are such that they plot above and to the right of the line of optimums shown in the lower plot on fig. 1.

Plots of unconfined compressive strength versus water content and density show the same pattern when plotted in the manner described above. Triaxial test results will also show the same pattern if the deviator stress at a low percentage of strain is used; they do not show the reduction in strength at the higher densities if the maximum or ultimate value of the deviator stress is used. Unconfined and triaxial test results are presented in the published reports; however, only the CBR test results are treated in this paper.

The decrease in CBR with an increase in density under certain circumstances has also been observed in field test sections at the Waterways Experiment Station. The most striking occurrence was in a series of test sections constructed to test the life of steel landing mat under a heavy airplane wheel load. The subgrade was the same as the soil described previously. The test sections were intended to have a CBR of 15 per cent, and the first test section, constructed at an average water content of 16.9 per cent, yielded CBR values in the desired range. As traffic was applied, however, the CBR decreased to well below the desired value of 15 per cent. The test was not entirely successful because failures developed too early. In the second test section, the soil was placed at a water content of 14.7 per cent, and the subgrade CBR increased during traffic to well above the desired value of 15 per cent. Again the test was not entirely successful because no failures developed during traffic. In the third test section, the average water content was 15.2 per cent. The CBR during traffic was generally in the desired range and the tests were successful because part of the track failed and part did not. The reduction in strength that occurs under certain conditions in the laboratory and in these field tests has been described in detail in an earlier paper.(1)

The CBR test results shown on fig. 1 were made on samples in the as-molded condition. Therefore, the pattern of results shown on these plates represents conditions immediately after compaction. The effect of increases in moisture after compaction can be studied by compacting specimens at a range of water contents and compactive efforts and subjecting the specimens to a soaking procedure before making the CBR tests. The Corps of Engineers uses a four-day soaking period as an estimate of the most severe condition that is likely to occur in nature (exclusive of frost action). The samples are confined during the soaking period by a surcharge equivalent to the weight of pavement and base that will be above the material when construction is completed.

Typically, the soaking tests on the soil discussed above produce a large reduction in the CBR of the materials compacted dry of optimum, a smaller change for those compacted at above optimum, and only a slight change for those compacted on the wet side of optimum. Fig. 3 shows the pattern of CBR versus density for the soaked condition at equal molding water contents. The term "molding" should be emphasized because the molding water content has as much or more influence on the CBR than the water content reached in the soaking test. The low CBR for samples compacted both dry and wet and the higher CBR values for samples compacted at about optimum are well illustrated. The effect of the soaking procedure may be better illustrated by comparing results of tests for the as-molded and soaked conditions. Fig. 4



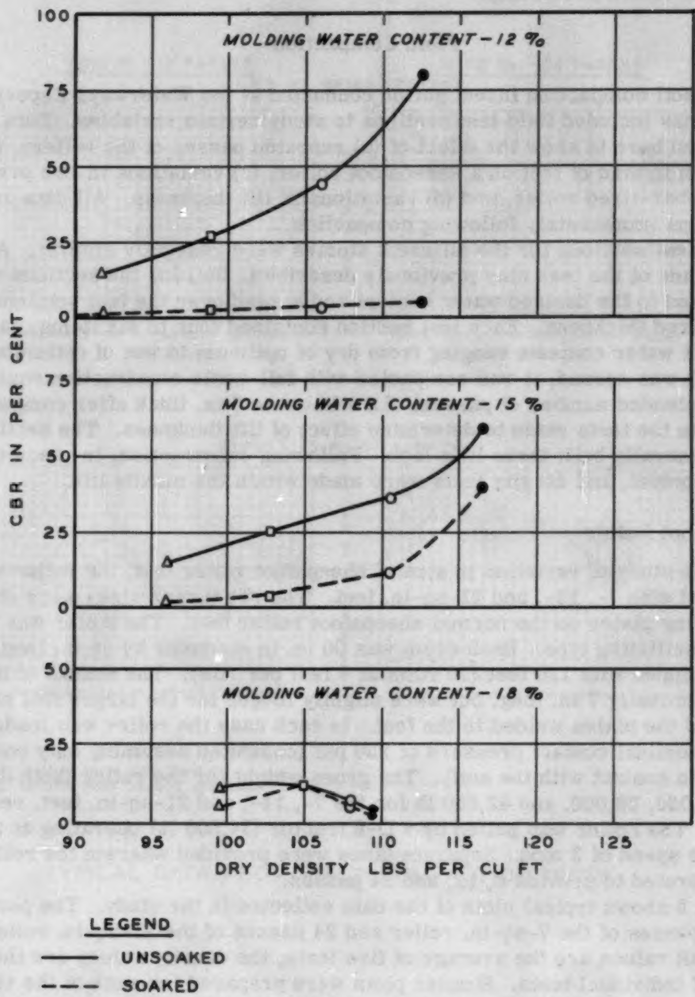
NOTE: FIGURE BESIDE CURVE IS
MOLDING WATER CONTENT.

EFFECT OF COMPACTION ON CBR

LABORATORY COMPACTION

SOAKED

FIGURE 3



EFFECT OF SOAKING ON STRENGTH

FIGURE 4

shows this comparison for three selected water contents, and exemplifies the trend described previously.

Field Compaction

The soil compaction investigation conducted at the Waterways Experiment Station has included field test sections to study certain variables. Data are presented here to show the effect of (a) repeated passes of the rollers, (b) variation in size of feet on a sheepsfoot roller, (c) variations in tire pressure on a rubber-tired roller, and (d) variations in lift thickness. All data are for conditions immediately following compaction.

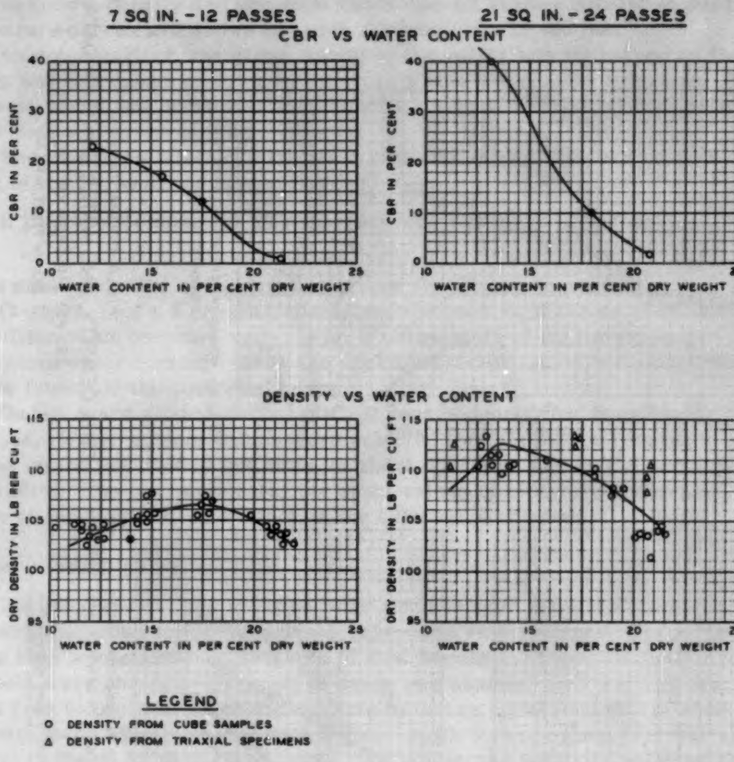
The test sections for the different studies were generally similar. All were made of the lean clay previously described. Soil for the sections was processed to the desired water content and spread over the test sections to the desired thickness. Each test section contained four to six items, each at different water contents ranging from dry of optimum to wet of optimum. As each lift was spread, it was compacted with full-scale construction equipment to the intended number of passes. All lifts were 6 in. thick after compaction except in the tests made to determine effect of lift thickness. The sections were generally built three lifts high. Following construction, in-place CBR, water content, and density tests were made within the middle lift.

Sheepsfoot Roller

In the study of variation in size of sheepsfoot roller feet, the roller was operated with 7-, 14-, and 21-sq-in. feet. The two larger sizes were obtained by welding plates on the normal sheepsfoot roller feet. The roller was a dual-drum oscillating type. Each drum was 60 in. in diameter by 66 in. long, and was equipped with 120 feet (30 rows at 4 feet per row). The shanks of the feet were nominally 7 in. long, but were slightly longer for the larger foot size because of the plates welded to the feet. In each case the roller was loaded to give a nominal contact pressure of 250 psi (computed assuming only one row of feet in contact with the soil). The gross weight for the roller (both drums) was 14,000, 28,000, and 42,000 lb for the 7-, 14-, and 21-sq-in. feet, respectively. The roller was pulled by a D-8 tractor (34,500 lb) operating at an average speed of 3 mph. Separate lanes were provided wherein the roller was operated to provide 6, 12, and 24 passes.

Fig. 5 shows typical plots of the data collected in the study. The plots are for 12 passes of the 7-sq-in. roller and 24 passes of the 21-sq-in. roller. The CBR values are the average of five tests; the density values are the results of individual tests. Similar plots were prepared for each of the variables (nine plots each for CBR and density). The plots on fig. 5 show the best of the nine plots from the standpoint of scattering of data, 7-sq-in. foot, 12 passes, and average conditions from the standpoint of scatter, 21-sq-in. foot, 24 passes. The curves shown on the plots of density were first drawn to fit the data on the individual plots and then adjusted to provide the best fit to the entire group of plots. Adjusted curves of CBR versus water content and density versus water content are shown on fig. 6. Curves of density versus CBR for equal water contents are also shown on fig. 6. Similar plots were prepared for the 14- and 21-sq-in. rollers but are not shown here.

Analysis of the density and water content data plotted as in fig. 6 shows that an increase in the number of passes or an increase in foot size had the



TYPICAL DATA FROM FIELD COMPACTION TESTS

FIGURE 5

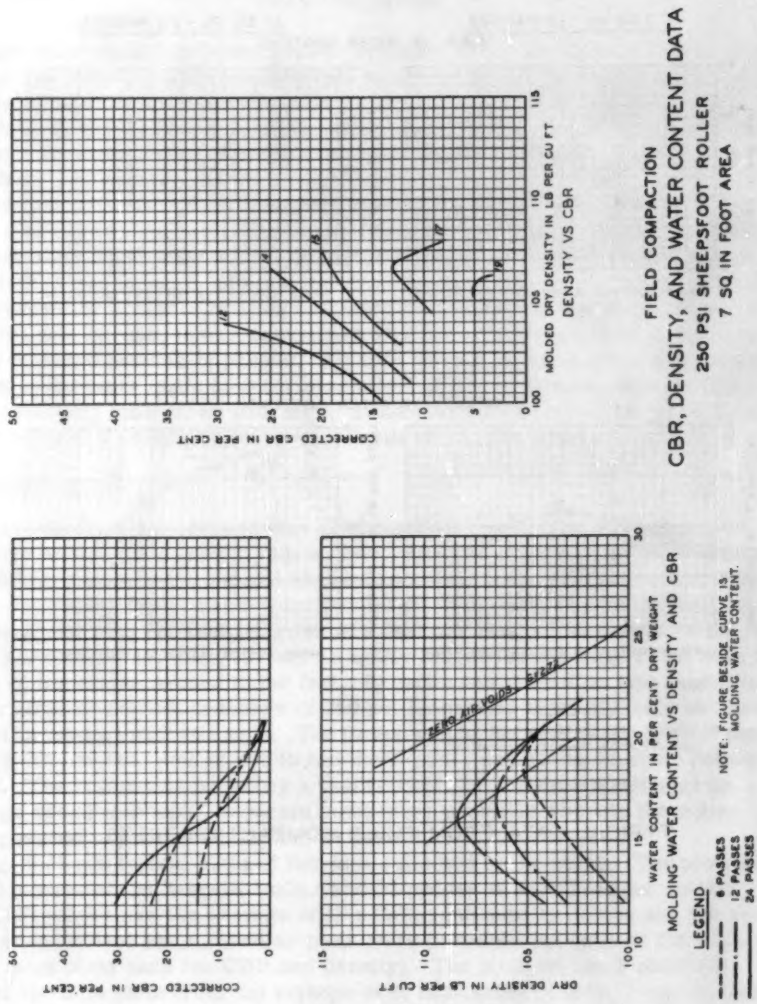

FIELD COMPACTION
CBR, DENSITY, AND WATER CONTENT DATA
250 PSI SHEEPSFOOT ROLLER
7 SQ IN FOOT AREA
CORRECTED CBR IN PER CENT
MOLDED DRY DENSITY IN LB PER CU FT
DENSITY VS CBR
CORRECTED CBR IN PER CENT
DRY DENSITY IN LB PER CU FT
ZERO AIR VOIDS
WATER CONTENT IN PER CENT DRY WEIGHT
MOLDING WATER CONTENT VS DENSITY AND CBR
NOTE: FIGURE BESIDE CURVE IS MOLDING WATER CONTENT.
LEGEND
6 PASSES
12 PASSES
24 PASSES

FIGURE 6

same effect as an increase in the compactive effort in the laboratory test. As the number of passes or the foot size was increased, the maximum density increased, and the optimum water content decreased. Fig. 7 presents plots of maximum density and optimum water content versus number of passes. Separate curves are shown for each different size of the feet.

As noted earlier, the gross weight of the roller was increased as the foot size was increased in order to maintain a constant contact pressure. The effects which are presented here as effect of foot size are a combination of foot size and gross weight.

The combined effect of changes in passes and foot size can be shown by use of the following combined factor:

$$E \text{ (index of compactive effort)} = \frac{\text{foot size x number of passes}}{42}$$

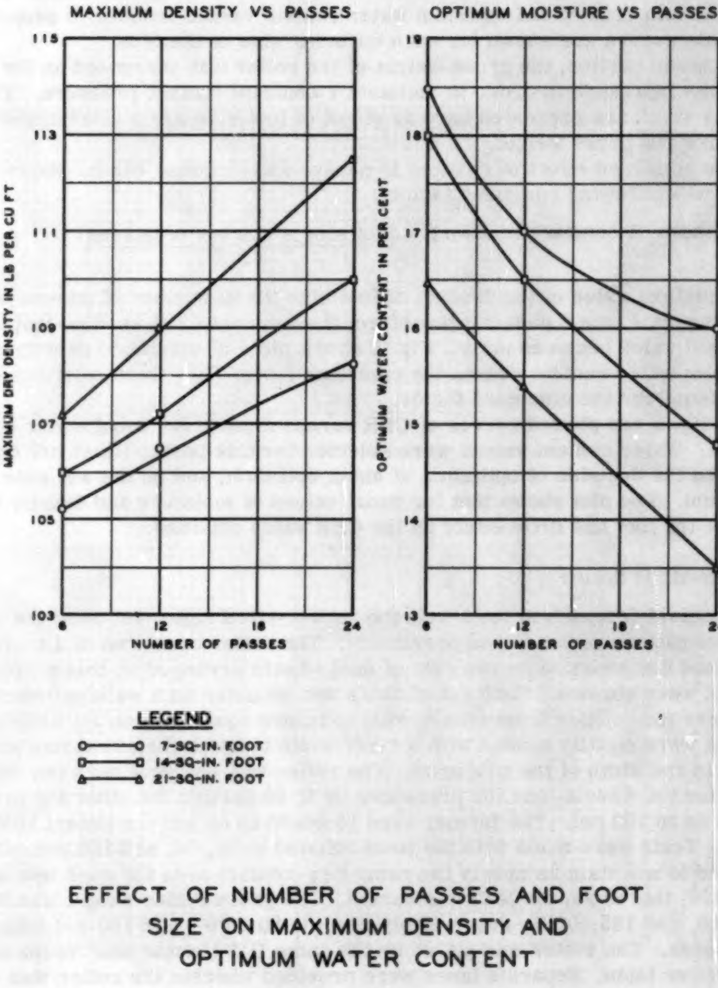
The smallest value of the product of foot size times number of passes equals 42 (7-sq-in. foot x 6 passes); therefore, the constant 42 is used so that the smallest value becomes unity. Fig. 8 shows plots of maximum density and optimum water content versus the combined factor (E). Good relationships were found for the combined factor.

On fig. 9 are plotted curves of CBR versus density for equal water content values. Water content values were selected for this plot to illustrate conditions on the dry side of optimum, at about optimum, and on the wet side of optimum. The plot shows that for equal values of moisture and density the size of the foot has little effect on the CBR value obtained.

Rubber-tired Roller

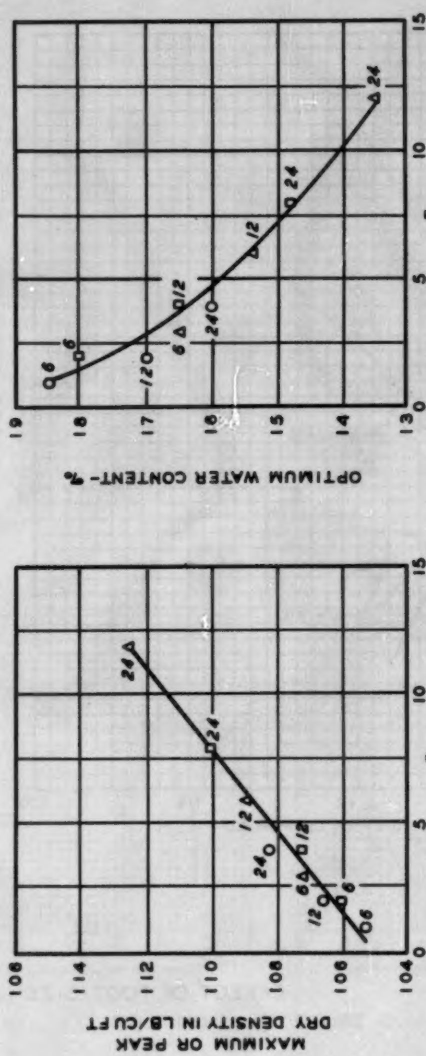
The field compaction tests with the rubber-tired roller followed the same general pattern as described previously. The roller consisted of a trailer-type load box mounted on two sets of dual wheels arranged so that all four wheels were abreast. Each set of duals was mounted on a walking beam which was free to oscillate from side to side to insure equal load on all wheels. The wheels were equally spaced with a clear width between them that was almost equal to the width of the tire print. The roller was equipped with two assemblies for the wheels, one for pressures up to 90 psi and the other for pressures up to 150 psi. The former used 18:00x24 tires and the latter, 16:00x21 tires. Tests were made with the tires inflated to 50, 90, and 150 psi. It was desired to maintain as nearly the same tire-contact area for each test as possible; therefore, the load was varied. The gross roller weight was 63,500, 100,000, and 125,000 lb, respectively, for the 50-, 90-, and 150-psi inflation pressures. The roller was pulled by the same D-8 tractor used in the sheepsfoot roller tests. Separate lanes were provided wherein the roller was operated for 4, 8, and 16 coverages. A coverage is defined as one application of a tire print to the entire area being compacted. Two passes of the roller produced one coverage.

The results of the CBR, water content, and density tests were plotted individually as described for the sheepsfoot roller tests. Composite curves of CBR versus water content and density versus water content were prepared. In these tests the effect of tire pressure was greater than the effect of repetitions, and the results were prepared accordingly. Fig. 10 shows the composite curves for 4 coverages. Separate curves are shown for the different tire pressures. Similar plots were prepared for 8 and 16 coverages but are



EFFECT OF NUMBER OF PASSES AND FOOT SIZE ON MAXIMUM DENSITY AND OPTIMUM WATER CONTENT

FIGURE 7



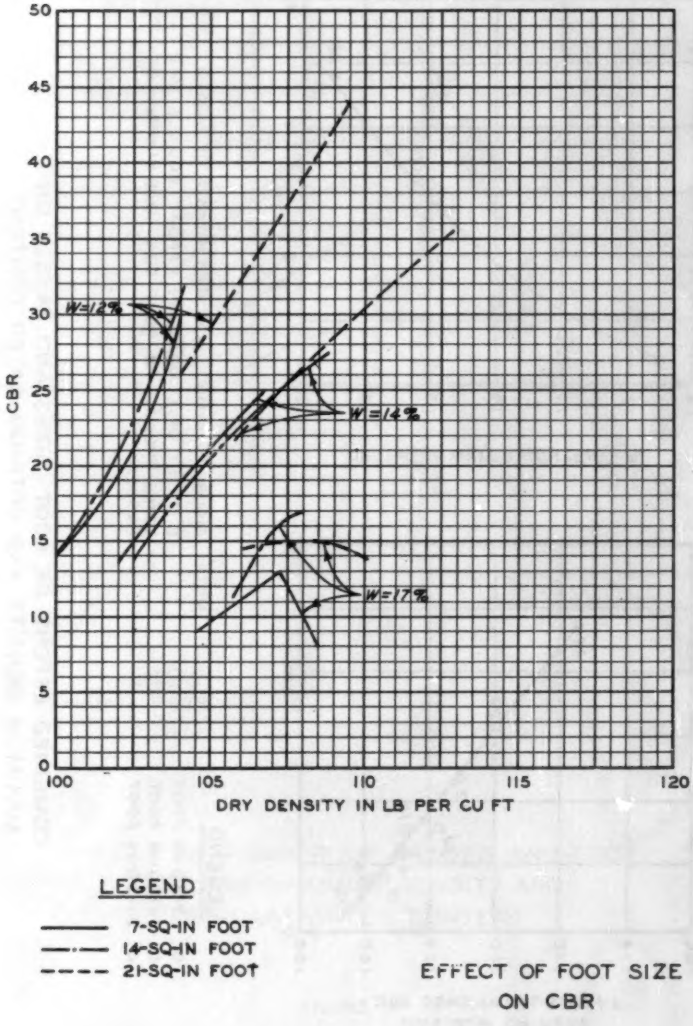
LEGEND

O 7-SQ-IN FOOT
□ 14-SQ-IN FOOT
Δ 21-SQ-IN FOOT

NOTE: FIGURE BY SYMBOL INDICATES NUMBER OF PASSES
E (INDEX OF COMPACTIVE EFFORT) EQUALS 1/42 TIMES PRODUCT OF FOOT SIZE AND NUMBER OF PASSES.

COMBINED EFFORT OF FOOT SIZES AND PASSES ON MAXIMUM DENSITY AND OPTIMUM WATER CONTENT

FIGURE 8



EFFECT OF FOOT SIZE ON CBR

FIGURE 9

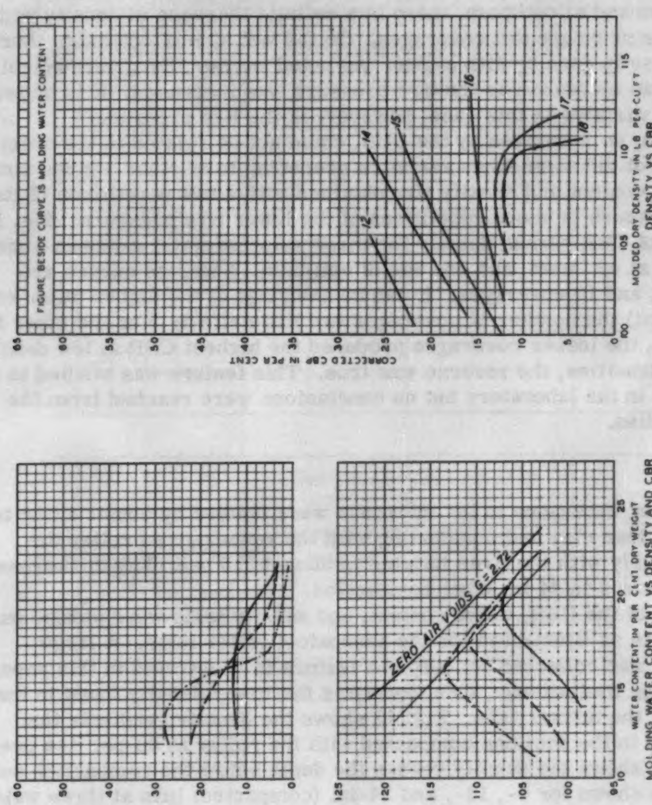


FIGURE 10

CBR, DENSITY AND WATER CONTENT DATA
LEAN CLAY TEST FILLS
4 COVERAGES RUBBER-TIRED ROLLER

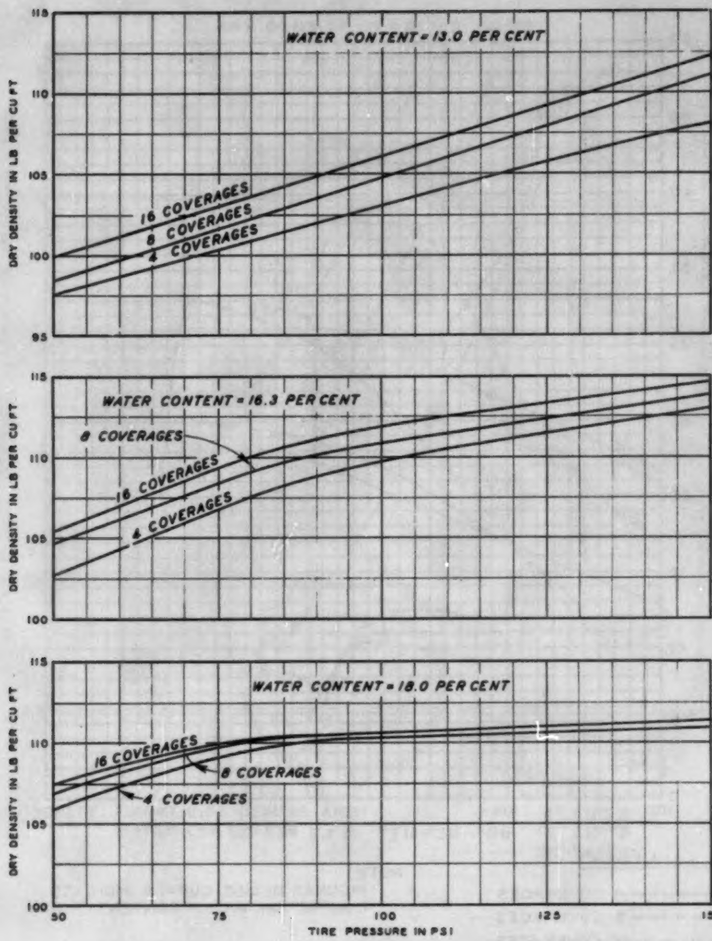
not shown here. It will be noted that an increase in tire pressure produces the same effect as an increase in compactive effort (number of blows) in the laboratory test. As the compactive effort is increased, the maximum density increases and the optimum water content decreases. The combined effect of tire pressure and coverages can be shown best by curves such as those shown on fig. 11. These plots show the maximum density versus tire pressure. Separate curves are shown on each plot for 4, 8, and 16 coverages. Plots are shown for a relatively dry water content, a water content at approximately optimum, and a water content wet of optimum. For water contents on the dry side of optimum and at optimum, there is a definite increase in density with increase in tire pressure and coverages. On the wet side of optimum, there is little increase in density with either. As noted earlier, the gross weight of the roller was increased as the tire pressure was increased. It is believed that the major variable in this case, however, is the tire pressure.

The strength, as measured by the CBR, could not be related to the specific tire pressure but did show some variation according to whether a given condition of water content and density was obtained with a few repetitions of the higher tire pressure or more repetitions of the lower tire pressure. Fig. 12 shows curves of CBR versus density for three selected water contents—one dry, one about at optimum, and one wet of optimum. Separate curves are shown for 4, 8, and 16 coverages. It can be seen that at the higher water content (18 per cent) the number of coverages had little effect. For the other two water contents, the lesser coverages produced the highest CBR at low densities. At high densities, the reverse was true. This feature was studied to a limited degree in the laboratory but no conclusions were reached from the laboratory studies.

Lift Thickness

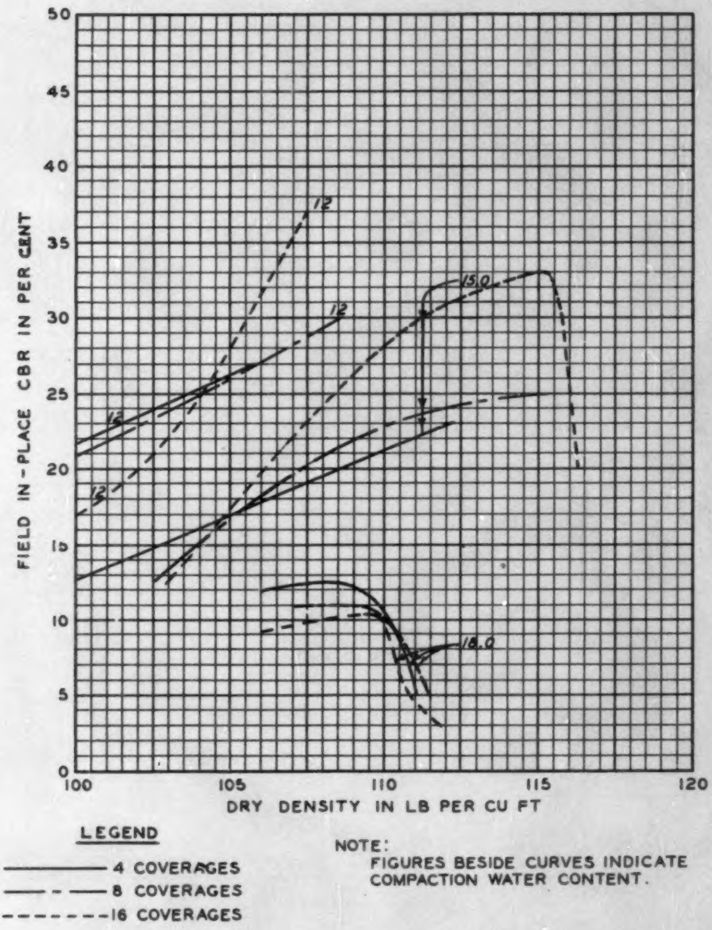
The effects of variations in lift thickness were studied by constructing test sections of the lean clay and compacting with the rubber-tired roller described previously with the tires inflated to 90 and 150 psi. Lift thicknesses varied from about 6 to 24 in. after compaction.

The results of the CBR, water content, and density tests were plotted on individual charts as illustrated for the sheepsfoot roller tests. A large volume of data was collected but only one feature is illustrated in this paper because of space limitations. This feature is the lower density found in the lower levels of the thicker lifts. Fig. 13 shows the density gradients that were measured in the sections compacted with the roller at 90-psi tire pressure. The plot shows the density versus the depth below the compaction surface. Data are shown for 6-, 12-, and 24-in. (compacted) lifts at three water contents. These data were interpolated from intermediate-type plots similar to those described previously. It is noted that near the compaction surface the density was about the same regardless of the lift thickness. The density decreased with depth below the compaction surface; the gradient was about the same in the 12-in. lift as in the 24-in. lift. It is apparent from these tests that thicker than normal lifts will result in a layered fill. Whether or not such conditions could be tolerated would depend on the specific purpose for which the fill was constructed. For fills that would be subjected to heavy wheel loads, as in airfield construction, the layered effect would not be tolerable because the repetitions of heavy wheel-load traffic of airplanes would produce compaction in the relatively uncompacted bottom zone of the thick



EFFECT OF TIRE PRESSURE AND
COVERAGES ON DENSITY

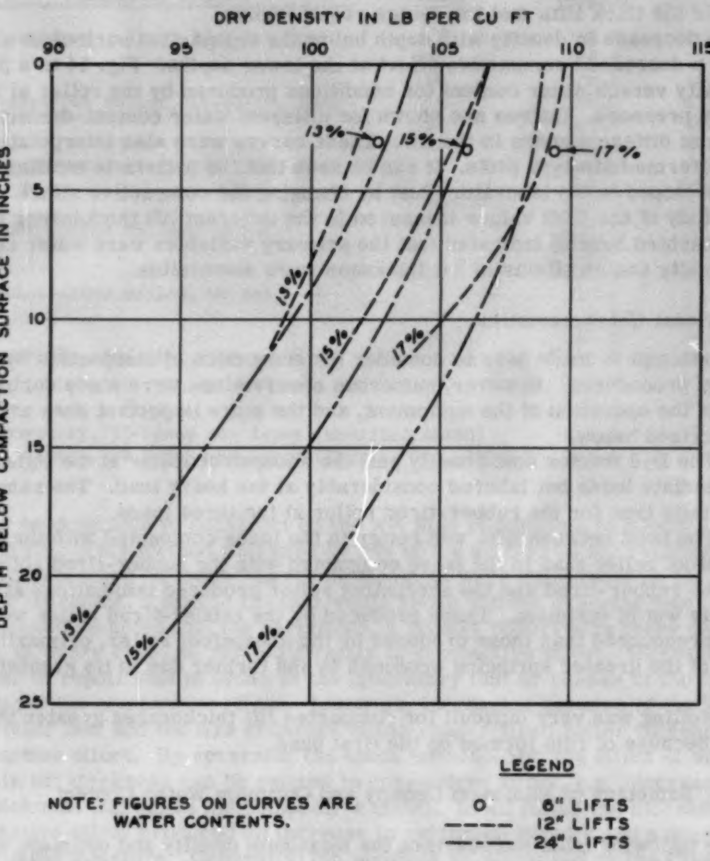
FIGURE 11



EFFECT OF COVERAGES ON CBR

LEAN CLAY TEST FILLS-COMPACTED WITH RUBBER-TIRED ROLLERS WITH TIRE PRESSURES OF 50, 90, AND 150 PSI

FIGURE 12



EFFECT OF LIFT THICKNESS
ON DENSITY

FIGURE 13

lifts that would result in objectionable settlement. The layered fill might be suitable in levee construction if the strength and permeability of the bottom zones of the thick lifts met the design requirements.

The decrease in density with depth below the compaction surface is a result of a decreased compactive effort at the lower depths. Fig. 14 is a plot of density versus water content for conditions produced by the roller at 90-psi tire pressure. Curves are shown for different water content-density conditions at different zones in the lift. These curves were also interpolated from intermediate-type plots. It can be seen that the pattern is similar to that developed in the laboratory test by changing the compactive effort.

A study of the CBR values measured in the different lift thicknesses (data not presented herein) indicated that the primary variables were water content and density and no effects of lift thickness were discernible.

Operational Characteristics

No attempt is made here to consider the economics of compaction by the various procedures. However, numerous observations were made during the tests of the operation of the equipment, and the more important ones are summarized below.

a) The D-8 tractor could easily pull the sheepsfoot roller at the light and intermediate loads but labored considerably at the heavy load. The same was essentially true for the rubber-tired roller at the three loads.

b) The bond between lifts was better in the lanes compacted with the sheepsfoot roller than in the lanes compacted with the rubber-tired roller. Both the rubber-tired and the sheepsfoot roller produced laminations at water contents wet of optimum. Those produced by the rubber-tired roller were more pronounced than those produced by the sheepsfoot roller, primarily because of the greater springing produced by the former due to its greater weight.

c) Rolling was very difficult for compacted lift thicknesses greater than 12 in. because of ruts formed on the first pass.

Summary of Maximum Density and Optimum Water Content

The following table summarizes the maximum density and optimum water content obtained with the sheepsfoot and rubber-tired rollers under the various conditions. Laboratory data are shown for comparison. (Table on following page.)

Discussion

From the preceding data, it is apparent that a wide range of strengths can be obtained by proper consideration of water content and density. The desired water content can be obtained by usual construction practices. The desired density can be achieved by applying the proper compactive effort.

The density, and strength, should be adjusted to the conditions of the job. There is no merit in obtaining unusually high densities and strength unless these are of benefit to the design. In airfield and highway work, relatively high densities are desirable to prevent subsequent compaction under the large number of load repetitions that will occur. For dams and levees, a lower density may produce the most economical and adequate design. In some cases

Compactive Effort	Passes	Maximum Density	Opt. Water Content
Sheepsfoot roller, 7-sq-in. foot	6	105.2	18.5
	12	106.5	17.3
	24	108.4	16.0
Sheepsfoot roller, 14-sq-in. foot	6	106.0	17.3
	12	107.2	16.5
	24	109.8	14.7
Sheepsfoot roller, 21-sq-in. foot	6	107.1	16.4
	12	108.9	15.4
	24	112.5	13.5
Rubber-tired roller, 50 psi	4	107.1	19.5
	8	107.4	19.2
	16	108.5	19.0
Rubber-tired roller, 90 psi	4	110.9	17.5
	8	111.5	17.0
	16	111.7	16.9
Rubber-tired roller, 150 psi	4	113.4	16.0
	8	115.2	15.4
	16	116.2	14.7
Laboratory, 55 blows per layer (Modified AASHO)		116.5	14.6
Laboratory, 26 blows per layer (Approx. Std. AASHO)		111.3	16.0

it may be desirable to construct on the wet side of optimum to produce a plastic fill. This method was used by one of the authors(2) for relatively low dams constructed on loess. Large settlements were anticipated in the foundation; therefore, a plastic fill that would settle without cracking was desired.

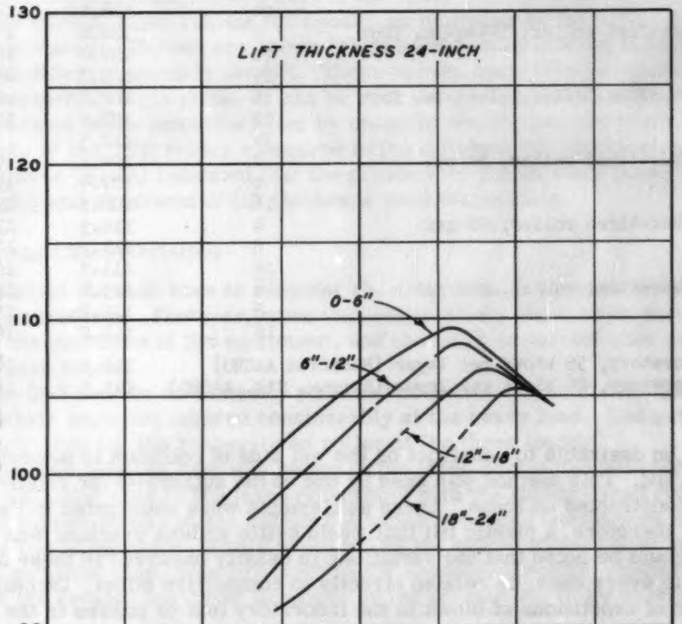
It should be noted that the variations in density observed in these data could, in every case, be related directly to compactive effort. Increasing the number of repetitions of blows in the laboratory test or passes in the field increased the compactive effort. Likewise, increasing the size of the sheepsfoot roller feet and the tire pressure of the rubber-tired roller increased the compactive effort. By reversing the usual terminology, the effect of variations in lift thickness can be related to compactive effort, e.g., decreasing the lift thickness increased the compactive effort. In all cases the increase in compactive effort produced an increase in maximum density and a decrease in optimum moisture. Generally, the strength as measured by the CBR followed the variations in water content and density. The relationship between CBR, water content, and density is rather complicated and can best be shown by plots, such as those on fig. 2, for conditions immediately after compaction, and on fig. 3 for conditions after soaking.

CONCLUSIONS

Based on the data and discussions presented in this paper, it is concluded that:

a) Cohesive soils can be stabilized to yield a given strength over a fairly wide range of values by a proper consideration of the water content and density.

b) The relationship of strength, water content, and density is complex but follows the general patterns illustrated on figs. 2 and 3.



NOTE: FIGURES ON CURVES INDICATE DEPTH BELOW TOP OF LIFT.

EFFECT OF LIFT THICKNESS
90-PSI RUBBER-TIRED ROLLER

FIGURE 14

c) Variations in load repetitions, foot size, tire pressure, and lift thickness produce variations in compactive effort.

d) An increase in compactive effort produces an increase in maximum density and a decrease in optimum water content.

REFERENCES

1. "Reduction in Soil Strength with Increase in Density," by Charles R. Foster, Separate No. 228, Proceedings, American Society of Civil Engineers, July 1953.
2. "Utility of Loess as a Construction Material," by W. J. Turnbull, Proceedings of The Second International Conference on Soil Mechanics and Foundation Engineering, Vol V, pp 97-103.

JOURNAL
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Proceedings of the American Society of Civil Engineers

THRUST LOADING ON PILES

James F. McNulty,[1] A.M. ASCE
(Proc. Paper 940)

SYNOPSIS

Lateral load tests were performed on two separate projects for the National Advisory Committee for Aeronautics, Langley Field, Virginia. The first project utilized concrete cast-in-place piles, heads not fixed, embedded in a medium dense silty-sand. The second project employed fixed-end timber piles embedded in a medium sandy clay. The field data and an approximate method of analysis are presented and discussed. Graphs of a preliminary nature indicating the relationship between deflection and load for various soil conditions are included.

INTRODUCTION AND DESIGN ASSUMPTIONS

The flight research hangar was designed by the Austin Company of Cleveland, Ohio, and was constructed in the years 1949 to 1951. The Haller Testing Laboratories of New York City served as the soil consultants and assisted in the supervision of the tests. A thrust of 4 kips per pile was considered allowable on the Raymond standard concrete pile with the provision that it be test loaded during construction.

The high temperature structural research laboratory is now under construction. The foundation design was the author's problem. The design was complicated by the action of three thrusts. These thrusts varied in magnitude from 500 to 1050 tons, acted in different directions, and varied in time duration from a fraction of a second to 36 seconds. It was calculated that (in conjunction with the use of batter piles) an allowable thrust of 3 kips on the vertical piles was required in order to keep the number of piles required for thrust equal to that required for the vertical loading. It was believed that the embedding of the heads of the timber piles 2 feet into the concrete was required to keep the deflection within the specified 1/16 inch.

Note: Discussion open until September 1, 1956. Paper 940 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 82, No. SM 2, April, 1956.

1. Civ. Engr., National Advisory Committee for Aeronautics, Langley Field, Va.

Soil Data

A. Free-end Concrete Piles:

The top 2 to 3 feet of each pile is embedded in a soft brown clay with sand and shells. The number of blows (N) per foot from the standard penetration test for this clay stratum is approximately four.

The remainder of the pile is embedded in a fine grey silty-sand with shells. The N for the silty-sand stratum is ordinarily equal to approximately 12 except when a 2- or 3-foot stratum containing a high percentage of shells is penetrated which increases N to approximately 30. By numerous borings taken on the field, it has been found that these "cemented" layers of shells are isolated pockets which may extend only a few feet; it is exceptional when one is encountered in taking a boring. Figure 1 shows a record of the boring logs including a sieve analysis.

B. Fixed-end Timber Piles:

The uppermost 9 feet of soil surrounding these piles is a medium clay (N = 6) containing a shallow stratum (approximately 1 foot thick) of soft black silt.

A 3-foot stratum of soft clay (N = 2) separates the medium clay layer from the silty-sand stratum (same as described above except that no cemented layers were noted). Figure 2 shows a sectional view of subsoil at the test location.

Test Procedure

A. Free-end Concrete Piles:

These piles were Raymond standard concrete piles (taper 1 in. in 30 in.), 20-gage sheet-metal shell, and unreinforced. After driving (for driving record, see table 1), the shells were filled with 3000-psi high early strength concrete and allowed to set for at least 72 hours before testing.

A 50-ton vertical load test was first made on each of these piles. Loads were applied in 5-ton increments and held until settlement ceased. A calibrated jack reacting against a loaded box was the loading method utilized, and the deflections were measured with Ames dials. The vertical test results are shown on figure 3.

Lateral loads were then applied to each of the piles in 2-ton increments until either a 1-inch deflection or a load of 24 kips was obtained as shown on figure 4. Test piles Nos. 2 and 3 were loaded vertically with 20 tons throughout their tests while test pile No. 1 was unloaded.

The test setup, shown in figure 5, was arranged as follows:

1) A 24-in. x 24-in. x 2-in. steel plate was placed against the side of the pile.

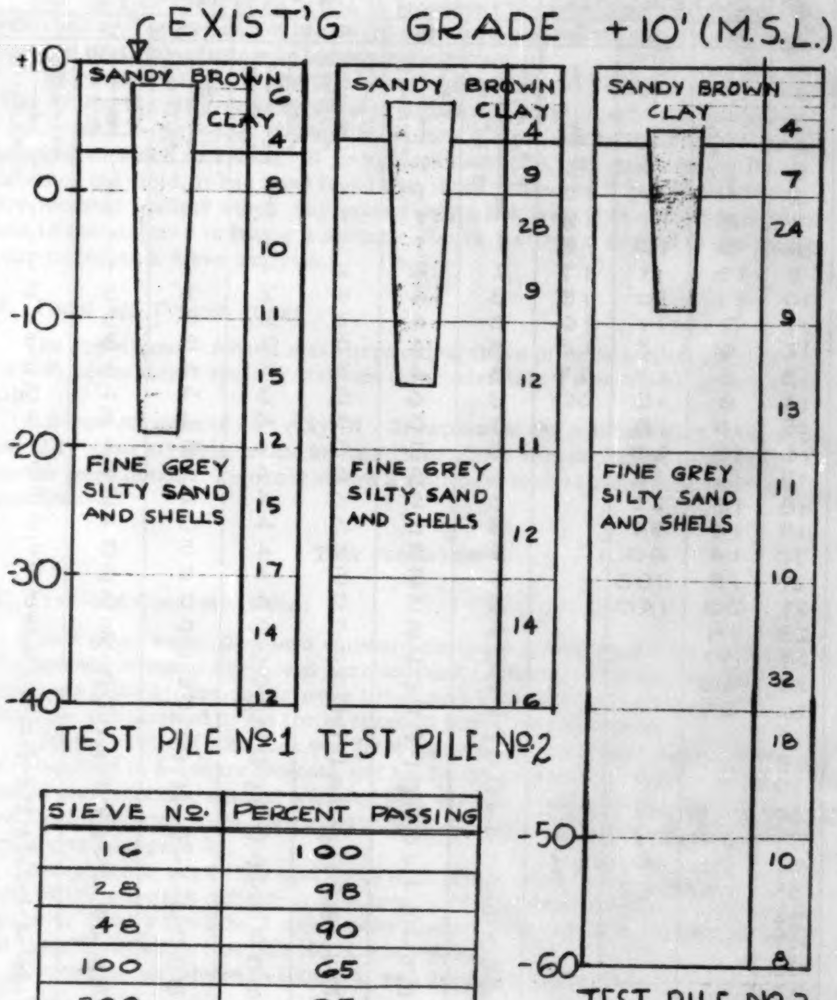
2) The jack was placed against the plate so that the load would be applied approximately 6 inches from the top of the pile.

3) A 12-in. x 12-in. timber with wedges was placed against the side of the excavation for the jack to react against.

4) Two 2-in. x 12-in. planks were placed so as to span the excavation over the center of the pile with their support far enough removed from the test area so as not to be disturbed.

TABLE I-DRIVING DATA, BLOWS/FT.

DEPTH	CONCRETE PILES			TIMBER PILES						
	1	2	3	4	5	6	7	8	9	10
1	RUN BY HAMMER			TIP = 8½" BUTT = 13"	TIP = 9" BUTT = 13"	TIP = 7" BUTT = 13"	TIP = 8" BUTT = 13"	TIP = 8½" BUTT = 13"	TIP = 8½" BUTT = 13"	TIP = 8½" BUTT = 13"
2										
3										
4										
5		6	5		1	1			1	1
6		8	10		1	1	1	1	1	1
7	8	11	13		1	1	1	1	1	1
8	8	10	18	1	1	1	1	1	1	1
9	13	9	17	2	2	2	1	2	1	1
10	12	10	15	3	4	3	2	3	3	2
11	7	11	16	3	4	3	2	3	3	3
12	6	12	18	3	4	3	2	3	3	3
13	8	13	23	3	4	3	3	4	4	3
14	9	15	30	3	6	5	3	4	4	5
15	15	18		5	6	5	4	4	5	5
16	16	20		5	5	6	4	4	4	5
17	11	22		6	5	6	4	4	5	5
18	13	25		5	5	6	4	4	5	5
19	14	34		5	5	5	4	5	4	5
20	14	40		5	5	5	4	5	5	5
21	15	108		5	5	5	4	5	5	5
22	20	140		5	5	5	4	5	5	5
23	19			6	5	5	6	6	5	5
24	39			6	6	5	5	5	5	5
25	40			6	6	5	6	6	6	5
26	39			6	6	6	5	6	6	5
27				6	6	6	5	6	6	5
28				7	6	6	6	6	6	6
29				6	6	5	6	6	6	6
30	NOTE: ALL PILES			7	6	7	5	7	6	6
31	DRIVEN BY A			7	8	6	6	8	6	6
32	VULCAN No. 1			7	7	6	6	7	7	7
33	HAMMER.			7	7	6	7	7	7	7
34				8	8	8	7	7	8	7
35				8	7	7	8	8	7	7
36				8	8	7	8	8	8	7
37				9	8	8	8	8	8	7
38				8	9	7	8	9	8	7
39				9	9	8		7	9	7
40				8	10	8		9	8	7
41				9	10	8		9	8	7
42				9	10	8		10	9	7
43				9	11	8		11	10	7
44				9	10	8		11	11	7
45				10	11	9		9	11	8
46				11	11	9		10	11	8
47				11	11	11		10	11	7



SIEVE No.	PERCENT PASSING
16	100
28	98
48	90
100	65
200	25
270	15

SIEVE ANALYSIS OF SILTY SAND

NOTE: WATER LEVEL AT TIME OF THE BORINGS WAS EL. PLUS 8 FEET.

Figure 1.

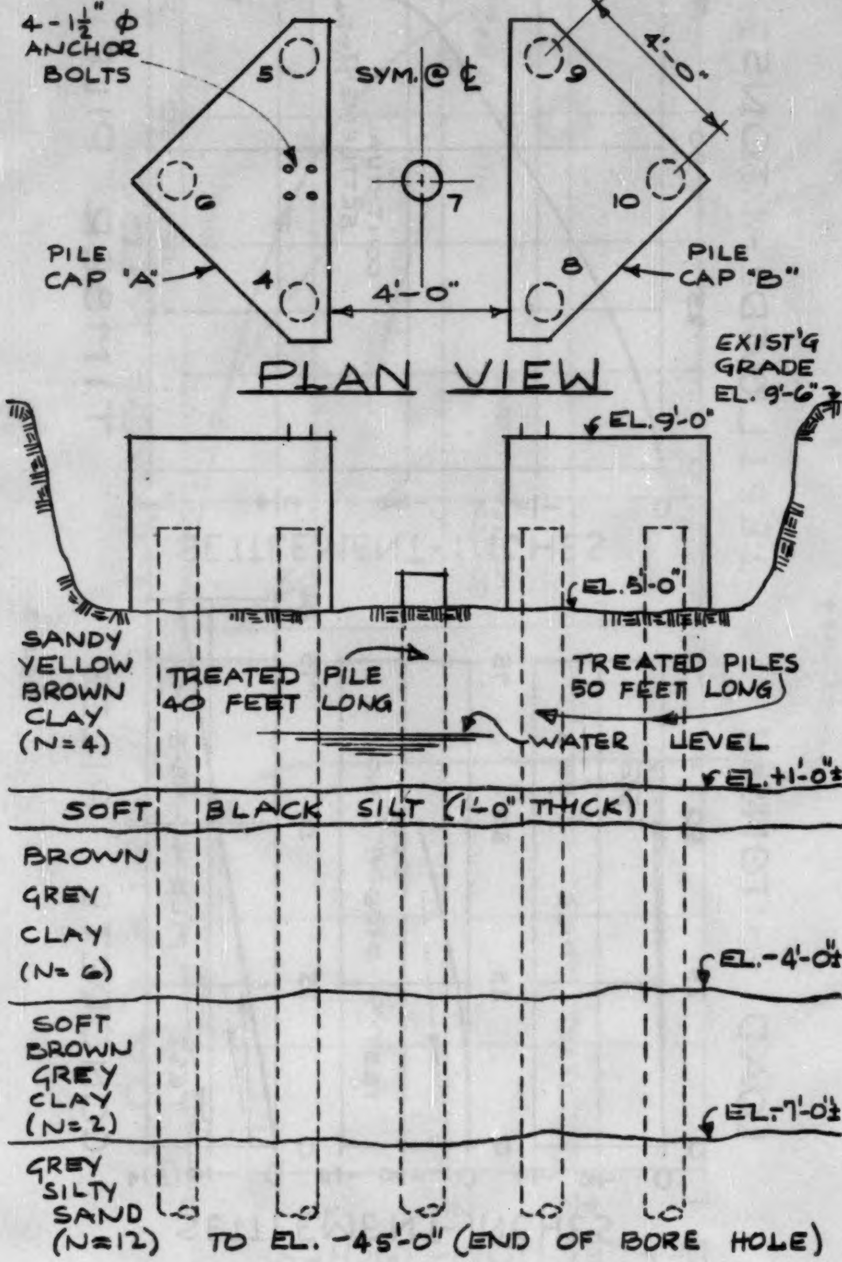

4 - 1½" Φ ANCHOR BOLTS
5
6
4
SYM. @ ℄
7
9
10
8
4'-0"
PILE CAP "A"
PILE CAP "B"
4'-0"
PLAN VIEW
EXIST'G GRADE EL. 9'-6"
EL. 9'-0"
EL. 5'-0"
SANDY YELLOW BROWN CLAY (N=4)
TREATED PILE 40 FEET LONG
TREATED PILES 50 FEET LONG
WATER LEVEL
EL. +1'-0"±
SOFT BLACK SILT (1'-0" THICK)
BROWN GREY CLAY (N=6)
EL. -4'-0"±
SOFT BROWN GREY CLAY (N=2)
EL. -7'-0"±
GREY SILTY SAND (N=12) TO EL. -45'-0" (END OF BORE HOLE)

Figure 2.

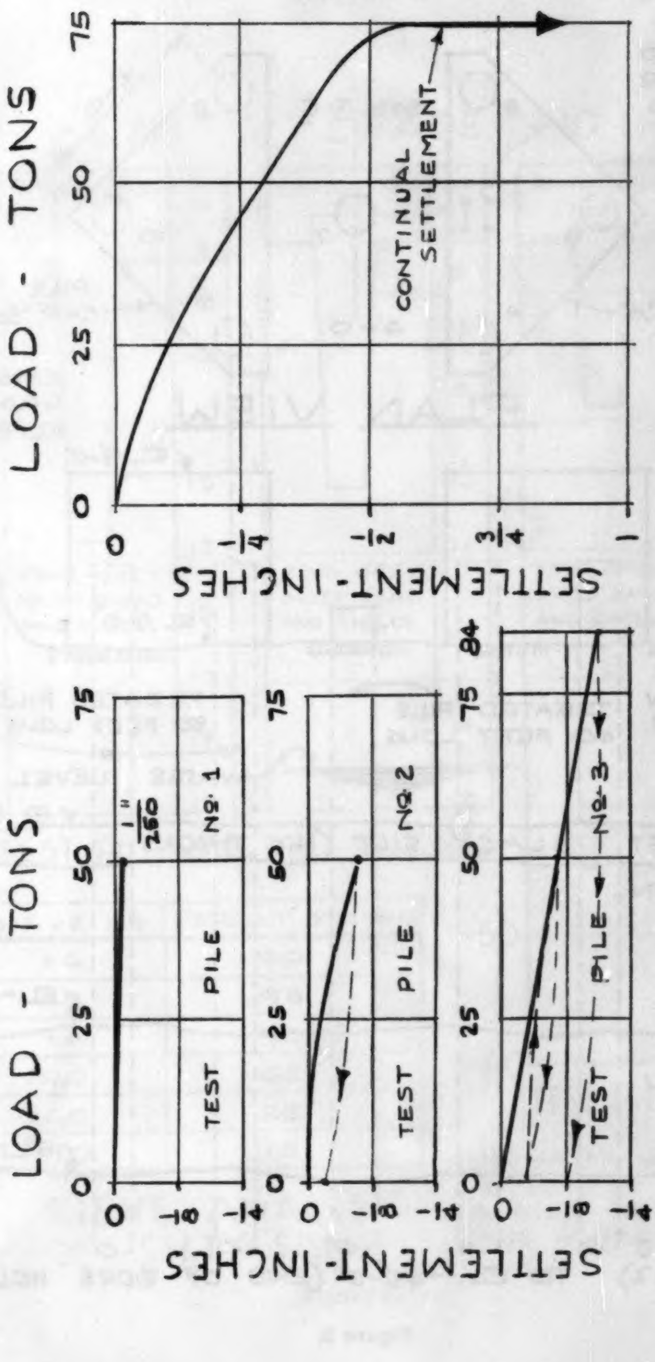

LOAD - TONS
SETTLEMENT - INCHES
CONTINUAL SETTLEMENT
TIMBER PILE
TEST PILE No. 1
TEST PILE No. 2
TEST PILE No. 3
CONCRETE PILES

Figure 3.

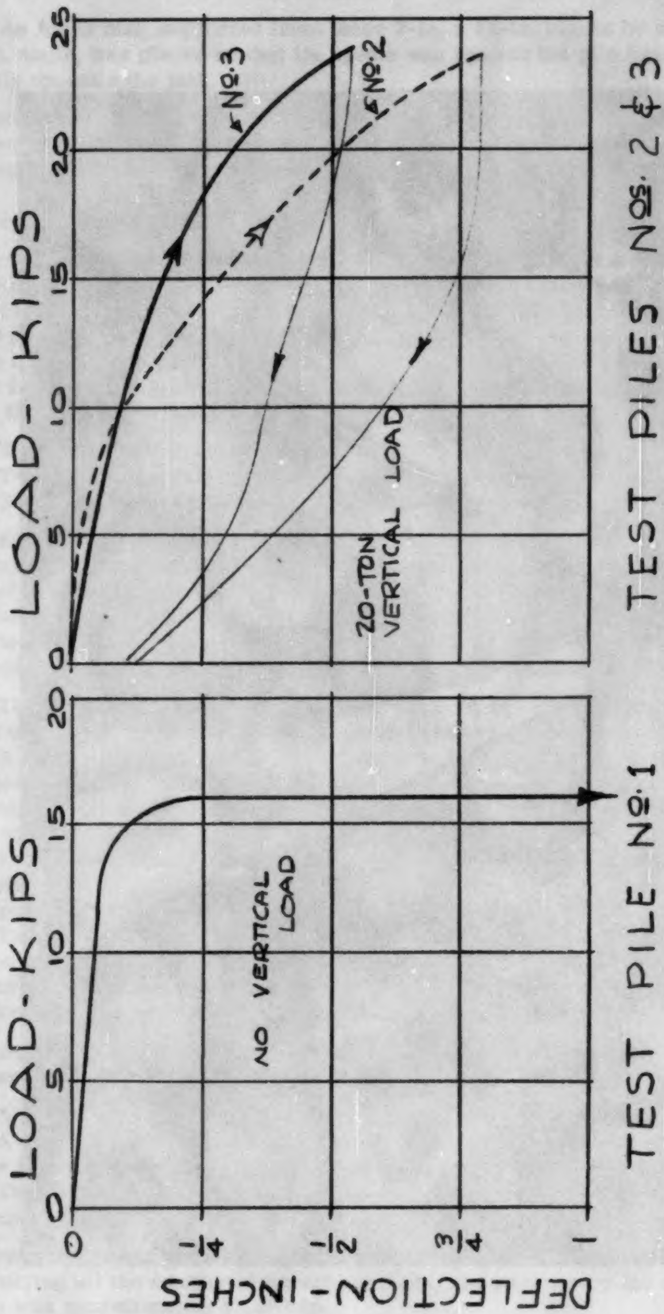

LOAD-KIPS
0
5
10
15
20
25
No.3
No.2
20-TON VERTICAL LOAD
TEST PILES Nos. 2 & 3
LOAD-KIPS
0
5
10
15
20
NO VERTICAL LOAD
TEST PILE No. 1
0
1/4
1/2
3/4
1
DEFLECTION-INCHES

Figure 4.

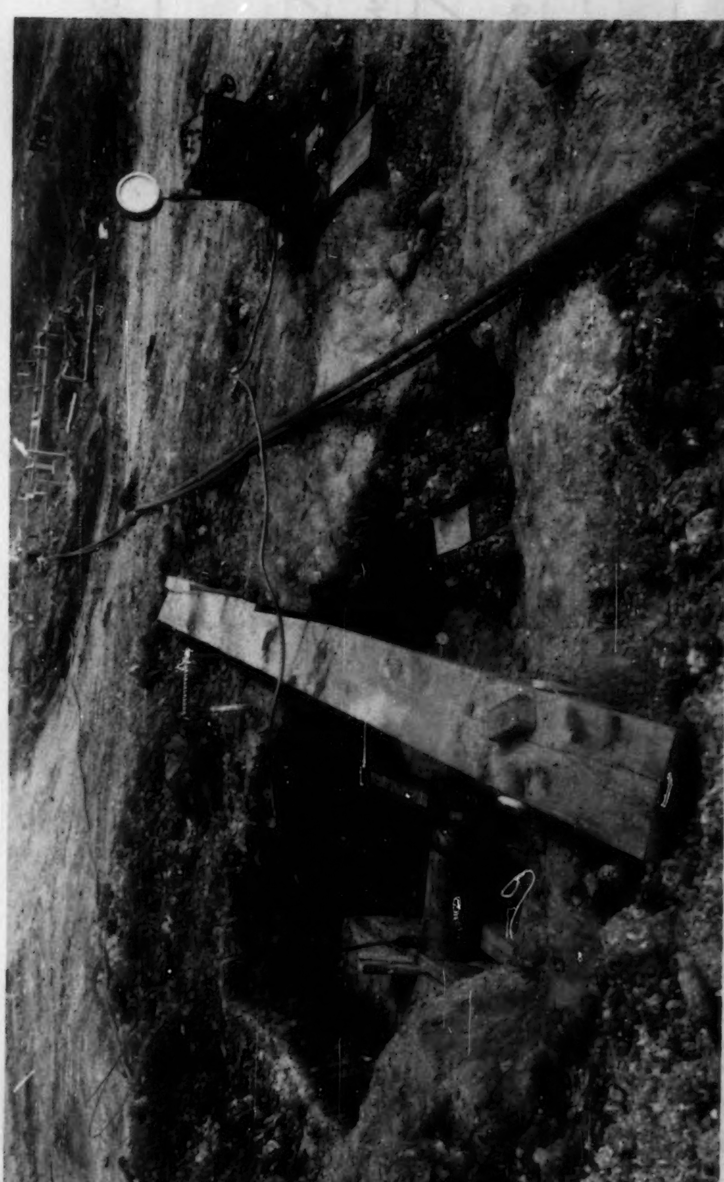


Figure 5.

5) An Ames dial, supported from these 2-in. x 12-in. planks by a 1-in. x 1-in. angle, was placed so that its needle was against the pile head directly opposite the jack.

Following the lateral test on test pile No. 3, another vertical load test was performed. This load test was carried to 84 tons which was the capacity of the test equipment and is plotted on figure 3.

B. Fixed-end Timber Piles:

These piles were southern yellow pine and were creosoted for a minimum retention of 12 pounds per cubic foot since the cutoff was several feet above the water table. The pile testing contract was let separately and prior to the project's pile driving but provision was made to space the test piles for their inclusion in the final project.

It was felt necessary that the lateral test be run on a cap containing three piles for the following reasons:

1) To insure against rotation of pile head
2) To give lateral stability against sidewise movement
3) To make extrapolation to the field condition feasible

Another three-pile cap was utilized for the jack's reaction; thus, in effect, two lateral tests were obtained. The piles were anchored against uplift and anchor bolts were placed in the cap to allow for the making of a vertical load test on another timber pile midway between the caps after completion of the lateral load test.

The lateral load test was performed in the following manner:

1) The jack was alined so that the load was applied centrally on the pile cap at the height of the pile heads as shown in figure 6.

2) A string was stretched between the caps, approximately 1/8-inch clearance, so that any tipping of the caps would be noted by the cap coming into contact with the string. Level readings were also taken periodically as an added precaution.

3) Four Ames dials, 1-inch travel, were placed against the caps so as to ensure reliable readings. These dials were supported independently of the caps.

4) The load was then applied in increments of 5 kips per pile until a deflection of approximately 1 inch was obtained; the test results are plotted on figure 7. The load was released at a load equal to 10 kips per pile and the rebound noted before continuing.

The test arrangement was altered for the vertical test as shown on figure 8 and described below:

1) A 24WF76 was bolted to the pile caps and used to jack against.

2) A steel plate was placed over the pile head and used both as a base for the jack and as a level surface for the dial recording.

3) The vertical load was applied in 15-ton increments until the settlement exceeded 1 inch as shown on figure 3.

The contractor's bid price for supplying and driving the piles, pouring the caps, supplying all the equipment except the dials, and performing the two load tests was approximately $3,000.00.

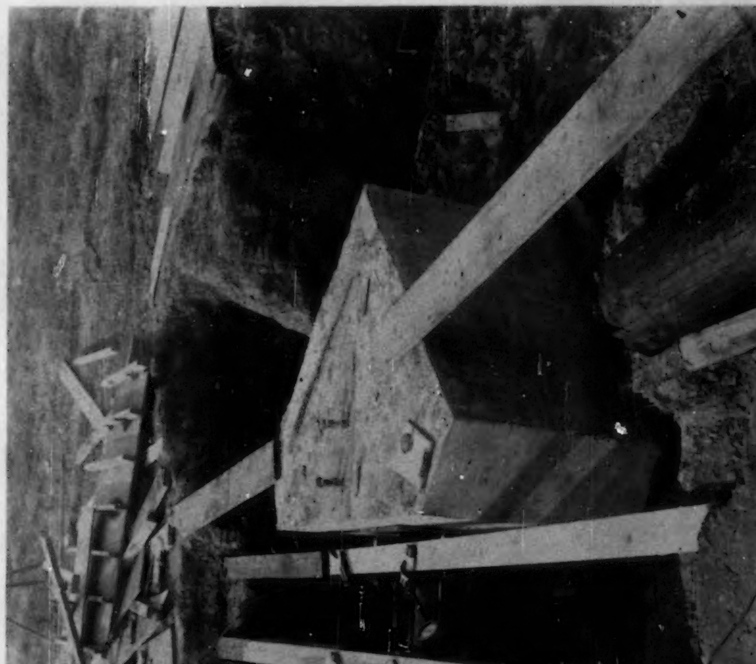


Figure 6.

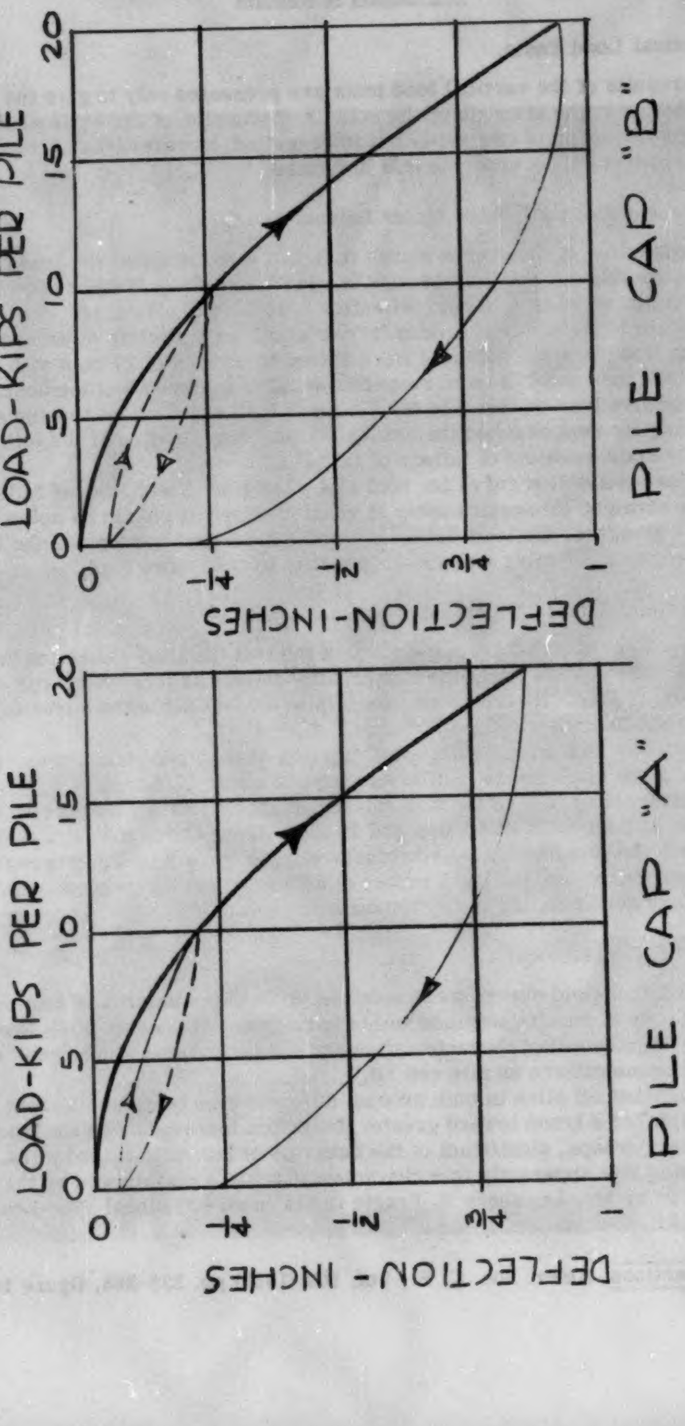

PILE CAP "B"
LOAD-KIPS PER PILE
DEFLECTION-INCHES
0
5
10
15
20
0
1/4
1/2
3/4
1
PILE CAP "A"
LOAD-KIPS PER PILE
DEFLECTION - INCHES
0
5
10
15
20
0
1/4
1/2
3/4
1

Figure 7.

Discussion of Results

A. Vertical Load Tests:

The results of the vertical load tests are presented only to give the reader some idea as to the strength of the soil. A discussion of the vertical pile tests is beyond the scope of this paper. It is regretted, however, that it was not feasible to test load a concrete pile to failure.

B. Free-end Concrete Piles Under Lateral Load:

An inspection of the curves shown in figure 4 reveals that the load deflection relationship for the first 10 kips is nearly linear. It further shows that the deflection at 10 kips is approximately 1/16 inch in all cases.

Test pile No. 1 suffered sudden failure at a thrust slightly in excess of 12 kips. As test piles Nos. 2 and 3 were loaded to thrusts of 24 kips with deflections of 3/4 inch and 1/2 inch, respectively, it is believed that the absence of a compressive load on test pile No. 1 caused it to crack on its tension side as its bending moment reached the critical value. Examination of the pile head gave no visible evidence of failure of the pile.

The load-deflection curve for both test piles Nos. 2 and 3 began falling off from the straight-line relationship at about 10 kips. It should be noted further that, in both cases, the load-deflection curve increased in slope as the load increased; this indicates a decreasing ability to take more load.

C. Fixed-end Timber Piles Under Lateral Load:

Figure 7 reveals that from the start of the test the load-deflection curves for these timber piles tend toward a greater deflection increment with each application of load. No linear relationship is evident since the curve is one of constantly increasing slope.

At a load of 10 kips per pile, the deflection of each cap was approximately 1/4 inch; at 20 kips, the deflection was approximately 7/8 inch.

It is interesting to note the pile rebounded approximately 80 percent of its deflection at both loads of 10 kips and 20 kips. This shows that at a deflection of 7/8 inch the pile had not failed structurally. Figure 9 is a photograph taken at extreme deflection; the top 2 inches of soil were muddy owing to rain the night before accumulating in the excavation.

D. General Conclusions:

It is felt that field checks were obtained as to the reliability of test data by the similarity of results obtained within each group. Concrete piles Nos. 2 and 3 displayed similar characteristics and the deflections of pile cap "A" followed the same pattern as pile cap "B."

The fact that all piles in both groups, different type piles in different type soils, displayed a trend toward greater deflection increments as the load increased is, perhaps, significant of the behavior of laterally loaded piles. It is worth noting that apparently this characteristic was also followed by the piles reported on by Mr. Lawrence B. Feagin in his paper[2] "Lateral Pile-Loading Tests."

2. Transactions, Amer. Soc. C. E., Vol. 102 (1937) pp. 236-288, figure 10(a).

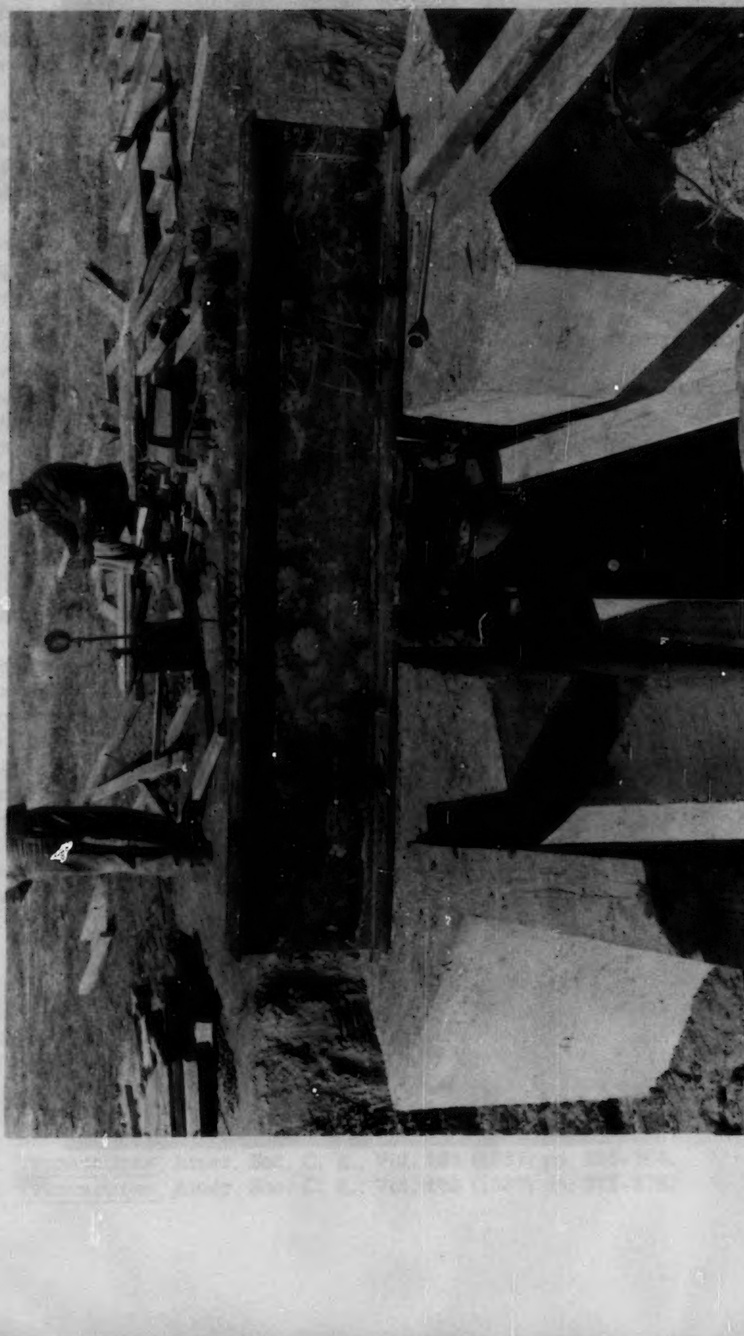


Figure 8.

Figure 9.

History of Theoretical Analysis

A. Solutions Available:

Only approximate methods of investigation are available for stresses, other than shear, since the three-dimensional elasticity problem of a load being applied to an elastic medium by an embedded rod has not been solved theoretically.

Mr. E. Titze[3] presented a solution in 1932 which served as a basis for later work of Mr. Cummings[4] and Mr. Chang[5] among others. Titze's solution was to treat the problem as a two-dimensional case of plane strain, that is, the pile is treated as a section of a continuous bulkhead and the problem is solved by considering the pile a bar supported on an elastic media of stiffness k. Titze assumed that $k = a_h Z^n$, Z = depth below grade, a_h and n = empirical constants where n = 1 for sand and n > 1 for clays. His work has had slight application due to the complexity of his resulting equations for deflection, shear, and moment.

Cummings and Chang have presented workable results by making various simplifications. Mr. Cummings assumed for piles in granular soil that $k = m_h Z$ (m_h is an empirical constant) and effected a solution by considering the pile fixed at a depth which minimized the energy of the system. Mr. Chang considered k a constant equal to E_s (modulus of elasticity of the soil). Theoretical curves for shear, moment, and deflection were obtained since the governing differential equation

$$EI \frac{d^4y}{dx^4} = \text{load} = -E_s y$$

readily lends itself to mathematical solution as long as E_s is constant.

Considerable progress has been reported in recent years in computing the deflection, moment, and shear along a pile's center line by means of difference equations. For purposes of identification, this difference equation solution method will be referred to as the Palmer-Brown method although it is realized that others, notably Messrs. J. B. Thompson and Sol M. Gleser, made significant contributions to its development. This method allows accurate theoretical curve fitting of the pile's deflection curve along its length once the parameters, k and n or their equivalents, are determined from full-scale field tests. A study of the Palmer-Brown method, contained in ASTM Special Technical Publications Nos. 154 and 154-A, is essential for an understanding of the lateral thrust problem.

B. Discussion of Solutions:

The accuracy of these solutions depends largely on the engineer's experience in assigning values to the empirical constants (a_h, n, m_h, or E_s) in the equations. No definite method has been yet advanced for a field or laboratory determination of these values. E_s as used above is not to be confused with

3. Widerstand des Pfahles Gegen Wagrechte Krafte, Dissertation (Tech. Hochschule Wien 1932).
4. Transactions, Amer. Soc. C. E., Vol. 102 (1937) pp. 255-264.
5. Transactions, Amer. Soc. C. E., Vol. 102 (1937) pp. 272-278.

"foundation modulus" which can be estimated from plate tests.

While it is true that an estimate of these empirical constants may be in error by 10 percent and only affect the final result by 2 percent since a fourth or fifth root is involved, it is not always possible to assure that one's estimate will be within 10 percent (or even 100 percent) due to paucity of information. Mr. Terzaghi[6] says of these constants, "The reader should always be mindful of the crude approximations which are involved in these equations . . . cannot be determined directly by laboratory or by small-scale field tests . . . the extrapolation from test results is essentially a matter of judgment."

No matter how closely the engineer may estimate the constants involved, he still faces the inherent restriction in the theory which gives a linear relationship between load and deflection. Since the load-deflection relationship is not a straight line as revealed by actual field tests, this means that the engineer must estimate k (or the empirical constants on which it depends) on the basis of the deflection with which he is concerned; that k may vary considerably with deflection will be shown later in this paper.

Thus, it is seen that the following represents the present state of solution of the action of a vertical pile under horizontal load:

1) There is no solution for the corresponding three-dimensional problem in elasticity.

2) It is necessary to treat the problem as a two-dimensional problem in plane strain as an approximation.

3) It is necessary to assume that the action of the earth can be represented by a series of independent springs.

4) It is necessary to estimate the spring force, (k), by extrapolation from tests and judgment.

5) It may be necessary to modify the results if there are different limiting values of deflection since the spring constant varies with deflection.

Proposed Method

A. Description:

The approximate method for analysis suggested herein conforms to the above steps but specifies that the spring force be considered a constant in all cases. This constant will be determined from the results of lateral load tests and is termed the "equivalent spring constant."

It is possible to set up the theory so that test deflections at pile head can be explained by a concept of an "equivalent spring constant," k, as readily as by a k which varies linearly with depth. It is only in the case of a pile embedded in sand that there is a possibility that a closer approximation to actual shears and moments can be obtained through the use of a depth-increasing k; an increasing k is perhaps indicated here since in plane strain the passive pressure increases with depth in a granular material.

This questionable refinement will be neglected in the interest of simplicity since it is felt that any exactitude has long since been lost. The concept of constant spring stiffness permits the ready use of standard texts on elastic beam theory for solutions.

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6. Theoretical Soil Mechanics, by K. Terzaghi, pp. 345-366.

B. General Solutions of the Differential Equation:

1) Derivation

The change in shear per incremental length of a bar supported by springs can be expressed as the difference in that length between spring load and the applied load (q)

$$\frac{dQ}{dx} = ky - q \qquad (a)$$

From the relationship of the derivative of the moment equalling the shear, we obtain

$$\frac{dQ}{dx} = \frac{d^2M}{dx^2} = ky - q \qquad (b)$$

The bending equation of a beam is

$$EI \frac{d^2y}{dx^2} = -M \qquad (c)$$

Differentiating (c) twice, for a constant EI we obtain

$$EI \frac{d^4y}{dx^4} = -\frac{d^2M}{dx^2} \qquad (d)$$

From (b) and (d) is obtained the differential equation for the deflection curve of a beam supported on an elastic foundation. Along the unloaded parts of the beam, the equation is

$$EI \frac{d^4y}{dx^4} = -ky \qquad (e)$$

Equation (e) has been solved mathematically to obtain relationships for deflection, moment, and shear. The solutions are available in any text on the subject of elastic beams. The equations given below are taken from Beams on Elastic Foundation.[7]

2) Definition of terms

y	deflection of pile at pile head (inches)
P	thrust load (kips)
EI	flexural rigidity of pile (kip inches^2)
k	equivalent spring constant (kip/inches^2)
x	distance along pile axis measured from pile head (inches)
M	moment on pile (inch kips)

7. Beams on Elastic Foundation by M. Hetenyi, University of Michigan Press, 1946.

Q shear on pile (kips)

$1/\lambda$ characteristic length (inches) where $\lambda = \sqrt[4]{\frac{k}{4EI}}$

3) Free-end condition

$$y_x = \frac{2P\lambda}{k}(e^{-\lambda x} \cos \lambda x)$$

$$M_x = \frac{P}{\lambda}(e^{-\lambda x} \sin \lambda x)$$

$$Q_x = -P\left[e^{-\lambda x}(\cos \lambda x - \sin \lambda x)\right]$$

$$y = \frac{2P\lambda}{k}$$

$$M_{max} = \frac{-Pe^{-\pi/4}}{\lambda\sqrt{2}} \quad \text{at} \quad x = \frac{\pi}{4\lambda}$$

$$Q_{max} = -P \quad \text{at} \quad x = 0$$

4) Fixed-end condition

$$y_x = \frac{P\lambda}{k} e^{-\lambda x}(\cos \lambda x + \sin \lambda x)$$

$$M_x = \frac{P}{2\lambda} e^{-\lambda x}(\sin \lambda x - \cos \lambda x)$$

$$Q_x = -Pe^{-\lambda x} \cos \lambda x$$

$$y = \frac{P\lambda}{k}$$

$$M_{max} = +\frac{P}{2\lambda} \quad \text{at} \quad x = 0$$

$$Q_{max} = -P \quad \text{at} \quad x = 0$$

5) Pile length

Since the above formulas have been derived on the basis of an infinite beam, it follows that the pile must be embedded in the soil a sufficient depth to allow the assumption to be a reasonable one. According to Hetenyi, the length required for the effect to become negligible is π/λ.

Design Procedure

A. Construction of Design Curves:

Figures 10 through 13 present load-deflection characteristics for concrete and timber piles, free- and fixed-end, in each of three soil conditions for which test data were available. The solid deflection curves are actual test results while the dashed lines were extrapolated or interpolated from the test data. An effort to construct the dashed curves theoretically was discontinued

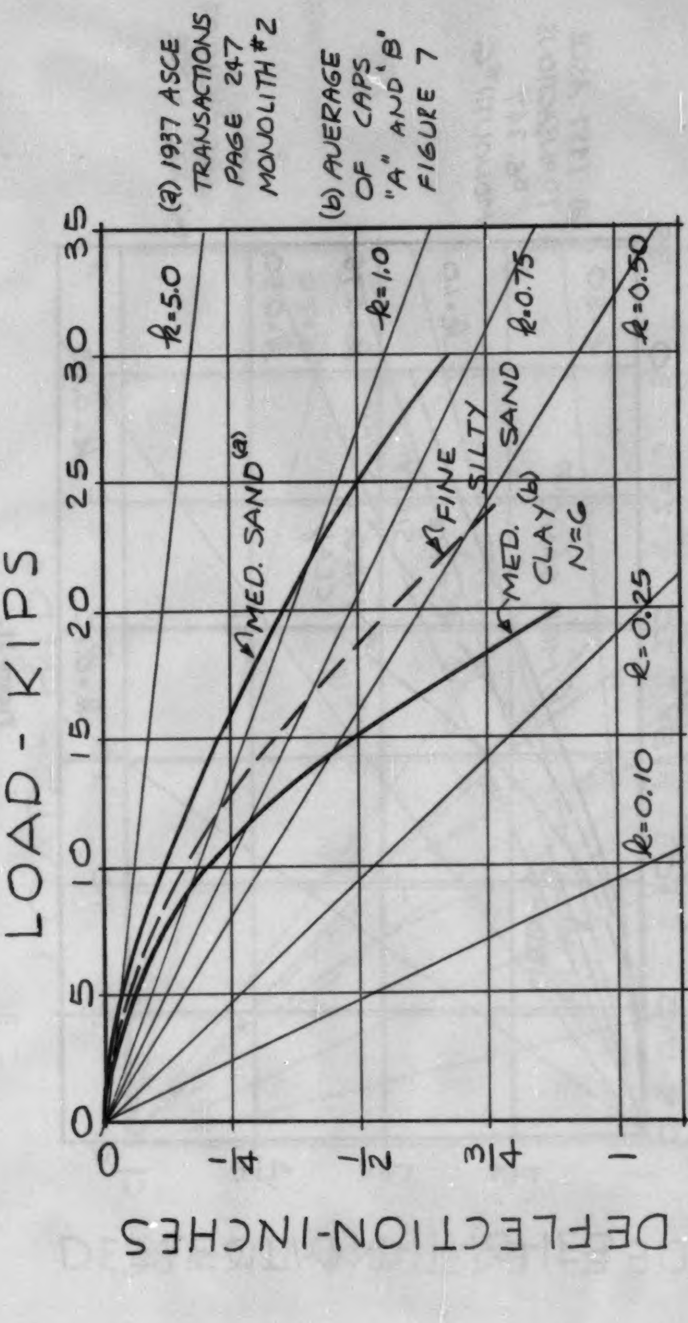
LOAD - KIPS
DEFLECTION-INCHES
0
5
10
15
20
25
30
35
1/4
1/2
3/4
1
(a) 1937 ASCE TRANSACTIONS PAGE 247 MONOLITH #2
(b) AVERAGE OF CAPS "A" AND "B" FIGURE 7
k=5.0
k=1.0
k=0.75
k=0.50
k=0.25
k=0.10
MED. SAND(a)
FINE SILTY SAND
MED. CLAY(b) N=6

Figure 10.

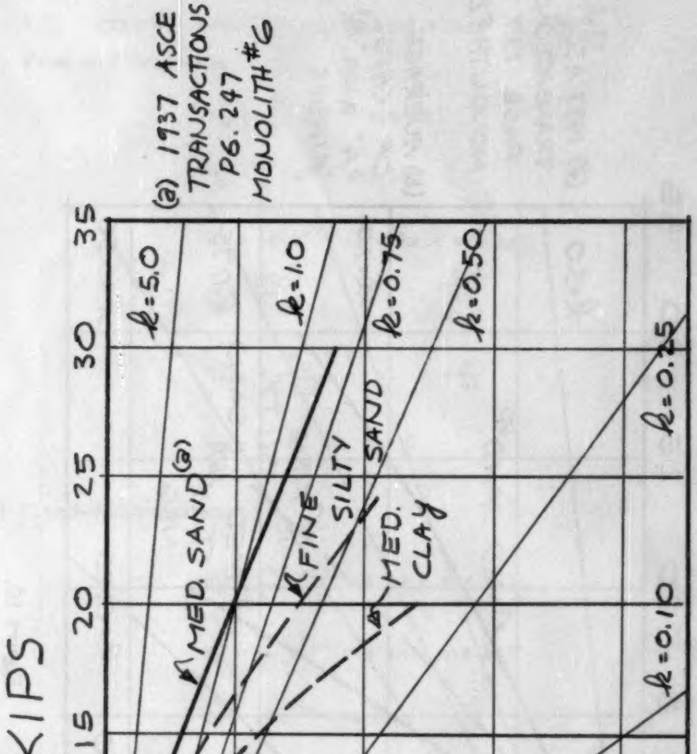
(a) 1937 ASCE TRANSACTIONS PG. 247 MONOLITH #6
LOAD-KIPS
0
5
10
15
20
25
30
35
DEFLECTION-INCHES
1/4
1/2
3/4
1
k=5.0
k=1.0
k=0.75
k=0.50
k=0.25
k=0.10
MED. SAND(a)
FINE SILTY SAND
MED. CLAY

Figure 11.

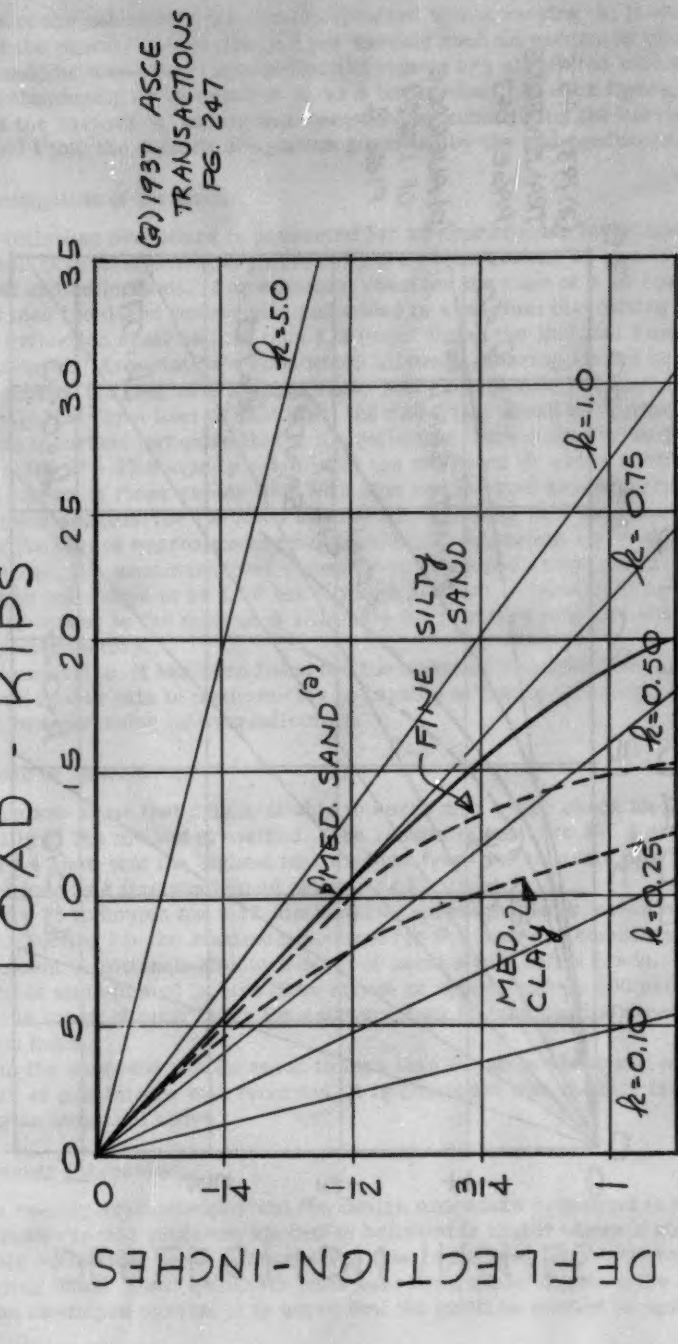
(a) 1937 ASCE
TRANSACTIONS
PG. 247
LOAD - KIPS
0
5
10
15
20
25
30
35
DEFLECTION-INCHES
0
1/4
1/2
3/4
1
k=5.0
k=1.0
k=0.75
k=0.50
k=0.25
k=0.10
MED. SAND(a)
FINE
SILTY
SAND
MED.
CLAY

Figure 12.

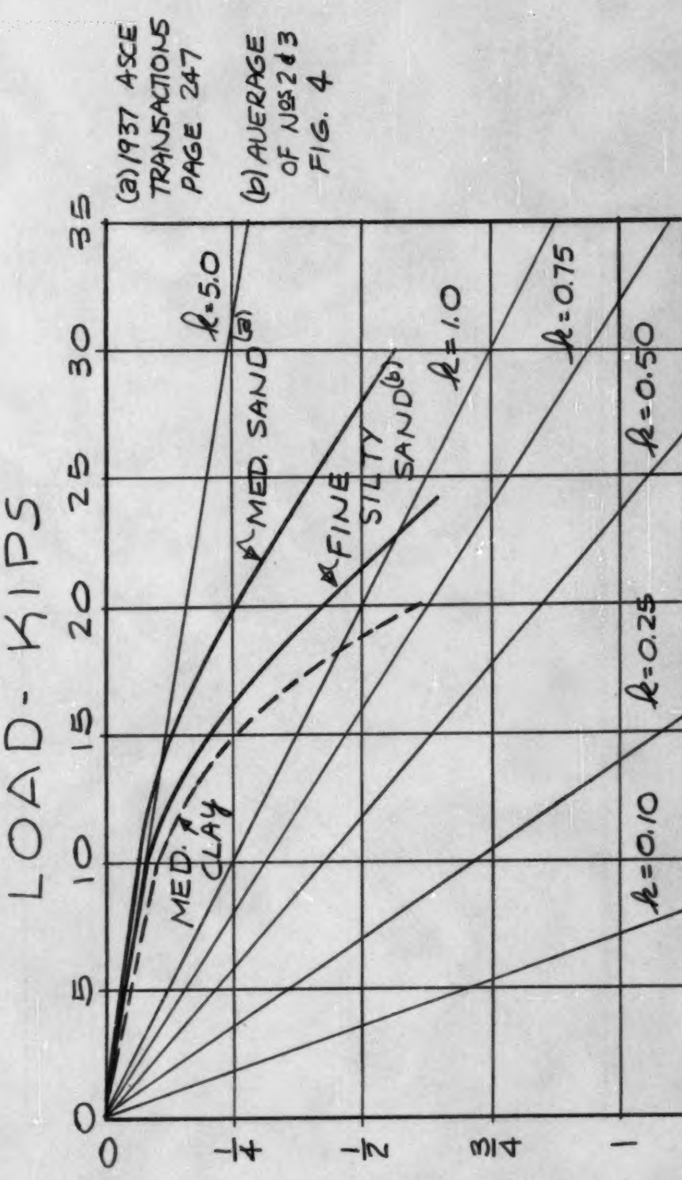
(a) 1937 ASCE TRANSACTIONS PAGE 247
(b) AVERAGE OF Nos 2 & 3 FIG. 4
LOAD- KIPS
0
5
10
15
20
25
30
35
DEFLECTION-INCHES
0
1/4
1/2
3/4
1
k=5.0
MED. SAND(a)
FINE SILTY SAND(b)
MED. CLAY
k=1.0
k=0.75
k=0.50
k=0.25
k=0.10

Figure 13.

because of the calculation intricacies involved with a varying k; it was also felt that the paucity of test data did not warrant such an excursion into theory.

It should be noted that these deflection curves are all plotted with the linear relationship for a constant k as a background. In each figure, the slope of the various k values was computed by substituting the corresponding E and I into the deflection equation governed by the end conditions.

B. Investigation of Stresses:

The following procedure is presented for an approximate investigation of stresses. It is first necessary to specify the requirements as to allowable stresses and deflections. For example, consider the case of a 20-ton vertically loaded fixed-end timber pile embedded in a medium clay where the lateral deflection shall be less than 1/2 inch. Using the National Lumber Manufacturers' Association's code which allows a shearing stress in southern yellow pine of 120 psi, an allowable shear load of 13.6 kips is found. Figure 10 reveals that for a load of 13.6 kips, the deflection would be approximately 3/8 inch; it further indicates that if the deflection were linear from the origin to this point (P = 13.6 kips, v = 3/8 inch) the indicated k value would be 0.60 ksi. Substituting these values (P = 13.6 kips and k = 0.60 ksi) into the governing moment equation for the fixed end condition results in a maximum moment at the butt of approximately 400 inch-kips. Assuming a butt diameter of 14 inches, the maximum fiber stress from combined compression and bending is calculated to be 1.75 ksi of which 1.50 ksi is from bending alone; this fiber stress is the maximum allowable for southern pine and will be considered satisfactory.

To summarize, it has been found for the assumed requirements a 20-ton fixed-end timber pile in medium clay is capable of taking a 13.6-kip thrust without overstressing or overdeflecting.

C. Check of Method:

There are some test data available to serve as a rough check as to the reliability of the method presented. The results of test pile No. 1 presented in figure 4 show that the highest test load the free-end concrete pile without vertical load in a fine sand could carry was 12 kips.

Figure 13 indicates for a 12-kip load that a reasonable k value would be 4.5 ksi. Solving for the maximum moment for the free-end condition results in a moment of 260 inch-kips at a depth of about 4 feet below grade. This moment gives an indicated failure fiber stress of approximately 500 psi which is within the range of probability for extreme fiber failure tension stress of a concrete beam.

Since the concrete was stressed to less than 90 psi in shear and no visual evidence of pile failure was recorded, it is concluded that the pile failed in bending as indicated above.

D. General Discussion:

It is readily acknowledged that the design procedure presented is not the final answer to this problem; all that is believed is that it offers a rough workable estimating basis although even this remains to be proven by substantiating tests. Until sufficient tests have been made to insure the reliability of the developed curves, it is urged that the outlined method be used in

conjunction with, and not in lieu of, field tests. In addition, if it appears as though the stress conditions may govern, it is recommended that the parameters be determined for use in the Palmer-Brown analysis as it is possible that the outlined method for stress investigation could give too conservative a result and, thus, be too expensive for use on a large project.

SUMMARY

The following conclusions are only tentative as is warranted by the scarcity of test data:

1) A Raymond standard concrete pile embedded in a fine silty sand with its head free to rotate developed an ultimate thrust resistance of approximately 25 kips at a lateral deflection of approximately 1 inch when insured against bending failure. Yield point was estimated at 15 kips.

2) A timber pile, 12 inches in diameter, embedded in a medium clay with its head fixed against rotation developed an ultimate thrust resistance of approximately 20 kips per pile at a lateral deflection of approximately 1 inch. There was no definite indication of a yield point.

3) The following thrusts are suggested as allowable for vertical piles under normal application and are based on a safety factor of 3 applied to the load required for a 1/4-inch deflection:

a) Free-end timber piles, 12-inch diameter

Medium sand	1500 pounds
Fine sand	1500 pounds
Medium clay	1500 pounds

b) Fixed-end timber piles, 12-inch diameter

Medium sand	5000 pounds
Fine sand	4500 pounds
Medium clay	4000 pounds

c) Free-end concrete piles, 16-inch diameter

Medium sand	7000 pounds
Fine sand	5500 pounds
Medium clay	5000 pounds

d) Fixed-end concrete piles, 16-inch diameter

Medium sand	7000 pounds
Fine sand	5500 pounds
Medium clay	5000 pounds

(It is noted that the same load results a 1/4-inch deflection in both the free-end and fixed-end concrete piles. It is felt that this is only a temporary contradiction which will be corrected by the reporting of more tests.)

4) Concrete piles which are used to resist thrust should be reinforced as required to resist bending stresses.

ACKNOWLEDGMENTS

The writer is indebted to Mr. Lawrence B. Feagin and the discussers of his paper Lateral Pile-Loading Tests both because the writer utilized the published test data and secondly because the technical matter contained therein has to a large degree influenced his thinking on the subject.

The results of the lateral pile tests on the free-end concrete piles have been published previously in the author's thesis entitled An Investigation Into the Compatibility of Theory and Load Tests for Foundations at Langley Field, Virginia presented to the University of Virginia as a partial requirement for the degree of Master of Applied Mechanics.

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JOURNAL
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

EARTHQUAKE RESISTANCE OF ROCK-FILL DAMS

Ray W. Clough[1] and David Pirtz,[2] Associate Members, ASCE
(Proc. Paper 941)

ABSTRACT

In this paper is described an experimental investigation of the effects of earthquakes on rock-fill dams with earthen cores. The tests were performed on 1/150 scale models of two types of dams, one having an inclined core near the upstream face, and the other having a central core. The models were subjected simultaneously to water loadings and to simulated earthquakes generated by the shaking table on which the models were constructed.

The test results show that rock-fill dams inherently are very resistant to earthquakes because of their flexible structure. No catastrophic slippages occurred in the models, even when subjected to ground accelerations exceeding the acceleration of gravity.

INTRODUCTION

A rock-fill dam may be defined as one in which the thrust of the impounded water is carried by an embankment of rock or gravel.(1)* The structure may be made watertight either by facing it with steel or concrete slabs, or by means of an impervious core of compacted earthen materials. This discussion will be limited to rock-fill dams of the latter type. Such dams may, in general, be divided into two broad classifications: those having a sloping impervious core near the front face, and those having a vertical core centrally located. Dams with sloping cores have certain advantages over dams with vertical cores in ease and economy of construction. However, because of uncertainties as to their stability when subjected to earthquakes, some doubt has been expressed as to the suitability of sloping core dams for use in the western regions.

Note: Discussion open until September 1, 1956. Paper 941 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 82, No. SM 2, April, 1956.

1. Associate Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.
2. Associate Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.

* Numbers in parentheses refer to References listed at end of the paper.

The purpose of this investigation was to determine qualitatively, by means of model tests, the effects of earthquakes upon these two types of impervious core rock-fill dams, and to compare their earthquake resistance.

The prototype of the sloping core dam was a preliminary design of the Kenney Dam, a structure about 300 feet in height recently constructed on the Nechako River in British Columbia. The prototype of the central core dam was a hypothetical structure similar to Mud Mountain Dam in Washington State, but scaled down to a height of 300 feet so that its dimensions would be comparable with those of the sloping core dam. Cross-sectional dimensions of the prototypes and models are shown in Figs. 1 and 2.

During the test program, models of the sloping core dam were subjected to simulated earthquakes of various intensities with the reservoir at three different levels: empty, 4/10 full, and full. The model of the central core dam was tested only in the full-reservoir condition. The simulated earthquakes ranged in intensity from maximum ground accelerations of less than 0.10 g (1/10 of the acceleration of gravity) to about 1.25 g.

Design of the Models

Similitude Requirements

In order that the results of the model tests might be representative of the earthquake response of the prototypes, it was necessary to consider the requirements of model similitude. In a structural model intended for dynamic loading, complete similitude is obtained if there is similarity between the model and prototype with respect to forces, lengths, and times. Length similitude is obtained by making the model geometrically similar to the prototype. Similitude of time is obtained if every event in the model is made proportional in duration to the corresponding event in the prototype. Similitude of forces requires that all forces in the model bear a constant ratio to the corresponding forces in the prototype. In general, the requirements of similitude in a geometrically similar model can be determined by consideration of the forces acting in the structure.(2) The time scale ratio may then be derived directly from the equations for similitude of dynamic forces.

In the discussion which follows, the various symbols will be defined as they are introduced. The subscripts $_m$ and $_p$ will be used to refer to the model and prototype, respectively.

The first step in the design of the models used in this investigation was the selection of a suitable length scale. The space available for testing limited the end-to-end length of the model to about 7 feet. Accordingly, the height of the model was established at 2 feet, since a length-to-height ratio of at least 3 was considered desirable to minimize the effects of end restraint. The ratio of model height to prototype height then determined the length scale λ. Thus

$$\lambda = \frac{L_m}{L_p} = \frac{2}{300} = \frac{1}{150}$$

where L represents any linear dimension, and all dimensions of the model were maintained in this scale.

The next step was determination of the model ratio of forces. Five types of forces were of importance in the analysis: (a) dead weight of rock-fill and

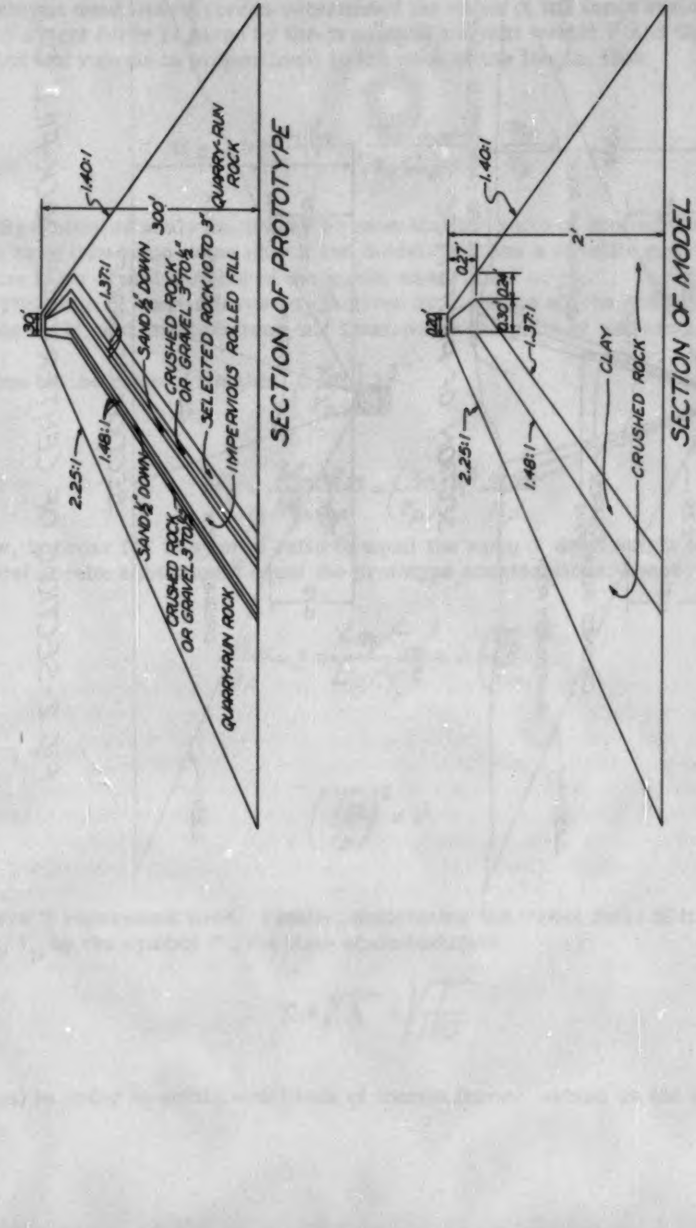


FIG.1-SECTIONS OF SLOPING-CORE ROCK-FILL DAM

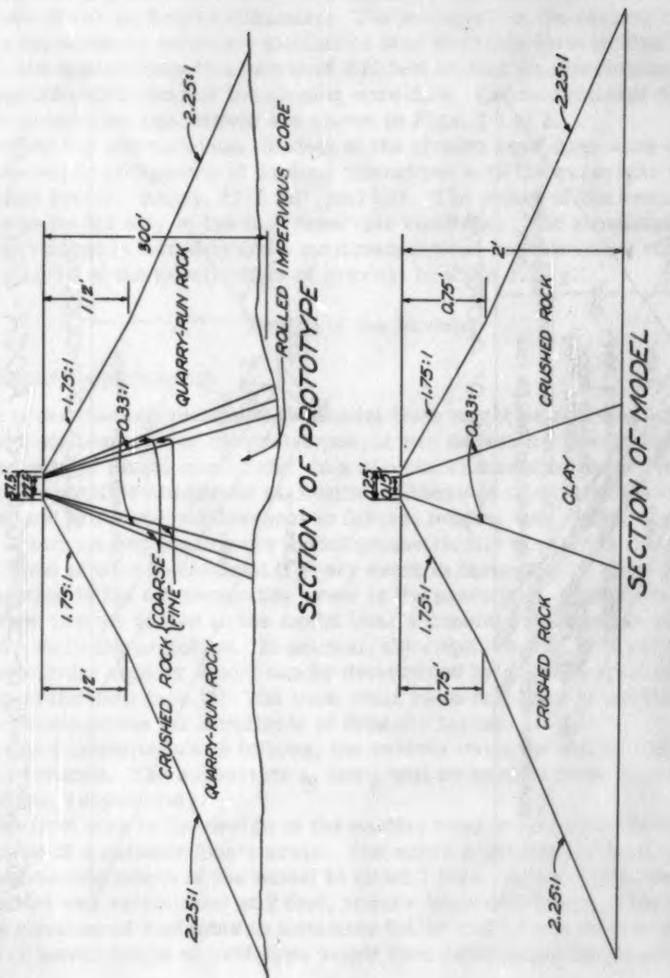


FIG. 2-SECTIONS OF CENTER-CORE ROCK-FILL DAM

core material, (b) applied water load, (c) inertia forces due to earthquake accelerations, (d) forces associated with elastic deformations in the structure, and (e) forces associated with the ultimate strength of the structure. For similitude to exist, the ratio of each model force to the corresponding prototype force must be the same for all types of forces. The force ratio will be designated by the symbol α with appropriate subscripts to indicate the type of force considered.

In this analysis, the ratio of the dead weight forces in the model to the prototype dead weight forces determined the value of the force ratio. The dead weight force is given by the product of the unit weight γ and the volume V, but the volume is proportional to the cube of the length, thus

$$\alpha_{D.W.} = \frac{\gamma_m V_m}{\gamma_p V_p} = \frac{\gamma_m}{\gamma_p}\left(\frac{L_m}{L_p}\right)^3 = \frac{\gamma_m}{\gamma_p}\lambda^3$$

By a similar analysis, it may be seen that the ratio of applied fluid forces will have this same value only if the model fluid has a specific gravity equal to the ratio of unit weights of the model materials γ_m/γ_p.

The ratio of inertia forces α_I is given by the ratio of the products of masses, M, and accelerations, a. Thus, since the ratio of masses is the same as the ratio of weights $\left(\frac{\gamma_m}{\gamma_p}\right)\lambda^3$

$$\alpha_I = \frac{M_m a_m}{M_p a_p} = \left(\frac{\gamma_m}{\gamma_p}\right)\lambda^3 \frac{a_m}{a_p}$$

Now, in order for this force ratio to equal the ratio of dead weight forces, the model accelerations must equal the prototype accelerations; hence

$$\frac{a_m}{a_p} = 1 = \frac{L_m/T_m^2}{L_p/T_p^2} = \lambda\left(\frac{T_p}{T_m}\right)^2$$

from which

$$\left(\frac{T_m}{T_p}\right)^2 = \lambda$$

where T represents time. Finally, designating the model ratio of time T_m/T_p by the symbol τ, the time scale becomes

$$\tau = \sqrt{\lambda} = \sqrt{\frac{1}{150}}$$

Thus, in order to obtain similitude of inertia forces, events on the model

(e.g., the periods of vibration) must take only 8.2% as long as the corresponding events in the prototype.

Forces associated with elastic deformations in the structure depend primarily upon the effective modulus of rigidity of the materials, since, in rock-fill dams, the only important deformations developed by earthquakes are shearing distortions. The force required to produce a given elastic strain is given by the product of the unit strain e, the modulus of rigidity G, and the cross-sectional area A; thus the ratio of forces due to elastic deformations α_E is given by

$$\alpha_E = \frac{e_m G_m A_m}{e_p G_p A_p}$$

In order to maintain geometric similitude, the strain in the model must equal the strain in the prototype; also, the ratio of areas is equal to the square of the length scale. Thus,

$$\alpha_E = 1 \frac{G_m}{G_p} \lambda^2$$

Equating this to the previously established force ratio yields the relationship,

$$\frac{G_m}{G_p} \lambda^2 = \left(\frac{\gamma_m}{\gamma_p}\right) \lambda^3$$

from which the ratio of moduli of rigidity required for similitude is found to be

$$\frac{G_m}{G_p} = \left(\frac{\gamma_m}{\gamma_p}\right) \lambda$$

Finally, in order for the model to simulate failures in the prototype, not only the force, but also the strength properties of the materials must be simulated. In other words, the model should fail at a force corresponding to the force which would produce failure in the prototype. The strength of the materials is assumed to depend upon two factors: the angle of internal friction and the cohesion. Similitude requires that the ultimate model force associated with each of these factors separately must be in the proper ratio to the corresponding prototype force.

Since the angle of internal friction is a dimensionless quantity, its magnitude is not affected by change of scale, and consequently internal friction forces can be simulated only if the angle of internal friction in the model is equal to that in the prototype. The ultimate cohesive force is given by the product of the cohesion C and the area A; thus the ratio of cohesive forces is given by

$$\alpha_C = \frac{C_m}{C_p} \frac{A_m}{A_p} = \frac{C_m}{C_p} \lambda^2$$

Equating this to the established force ratio yields the ratio of model to prototype cohesion required for similitude,

$$\frac{C_m}{C_p} = \left(\frac{\gamma_m}{\gamma_p}\right)\lambda$$

Selection of Model Materials

The relationships between model and prototype required by similitude having been determined, the next step in the design of the models was to select materials which would satisfy these requirements insofar as possible.

The dumped rock fill of the prototype was assumed to consist of essentially granular material ranging in size from sand to massive boulders and having an average unit dry weight of 110 pounds per cubic foot. The cohesion of this material was assumed to be negligible and the angle of internal friction was taken at 45°. (While this value is somewhat higher than normally used in design of rock-fill dams, its application here is justified by available test data as shown in Ref. 3.) For the rock fill of the model, a crushed quartzite was selected, of a size such that it would pass a 3/8" sieve and be retained on a No. 30 sieve. The unit dry weight of this material was 96 pounds per cubic foot; thus the unit weight ratio of the materials was

$$\frac{\gamma_m}{\gamma_p} = \frac{96}{110} = 0.87$$

and the force ratio α was

$$\alpha = \frac{\gamma_m}{\gamma_p}\lambda^3 = 0.87\left(\frac{1}{150}\right)^3$$

The value of the angle of internal friction of the model material as determined by direct-shear tests using relatively low normal loads was 48°; the value of the same angle as determined by vacuum triaxial test was 42°. Both of these values were considered to be sufficiently close to that of the prototype.

The clay core of the prototype was assumed to have a cohesion of about 1000 pounds per square foot and an effective angle of internal friction of 20°. However, the component of shear strength due to internal friction was neglected because of the possibility that pore pressures might be developed of sufficient magnitude to eliminate it. Thus, for similitude, a model material was required having a cohesion of

$$C_m = C_p \frac{\gamma_m}{\gamma_p}\lambda = 1000(0.87)\left(\frac{1}{150}\right) = 5.8\ psf$$

In addition, it was necessary that the material provide an impervious core in the model, and that it be stiff enough to be placed without difficulty during construction of the model. A commercially available kaolin clay was found to be suitable for the purpose. Because of its low plasticity, it was fairly easy to

mix and handle, and at a water content of 125 percent it had the required shear strength of 5.8 psf. The static shear strength of the clay, as determined by means of a quick undrained direct-shear test is plotted as a function of water content in Fig. 3.

It was considered advisable also to investigate the strength of this material when subjected to rapid loadings because of the fact that previous investigators had observed a considerable increase (40 percent to 200 percent) in the strength of saturated clays under such conditions.(4) Results of this study showed that under rapid loading (0.03 sec.) the kaolin mixed to a water content of 125 percent had a strength 70 percent greater than when subjected to a static loading (failure time of 5 minutes). Thus the model material may be considered to be similar to the prototype materials in this respect.

The effective modulus of rigidity of the materials used in the construction of rock-fill dams has been observed to be on the order of 12,000 pounds per square inch.(5) Thus in order to preserve similitude of dynamic deformations, the expected modulus of rigidity of the model materials should be

$$G_m = G_p \frac{\gamma_m}{\gamma_p} \lambda = 12000(0.87)\left(\frac{1}{150}\right) = 70 \text{ psi}$$

Based on the observed time required for the simulated earthquake shock to propagate through the height of the model, the effective modulus of the model dam materials was computed to be 120 psi or nearly twice as great as it should have been. The principal effect of this discrepancy in rigidity was to reduce the natural period of vibrations of the model from about 0.09 seconds as required by similitude to about 0.07 seconds. It will be demonstrated later that this reduction of period in the model would tend to increase the model stresses, and thus, the results obtained with the model should be conservative.

It was noted in the previous section that in order to maintain similitude of the applied loads, the specific gravity of the liquid loading the model should equal the ratio of the unit weights $\gamma_m/\gamma_p = 0.87$. While it might have been possible to obtain such a liquid, it was considered satisfactory to use water for loading the model. The effect of this discrepancy was to increase the loads on the model beyond the required value, and again the model results should be conservative.

Summary

The results of the similitude analysis and model design are summarized in the table on the following page. From this table it is seen that all required model values have been achieved with the exception of the density of liquid and the modulus of rigidity of the model materials. As was noted in the table, the discrepancy in both of these items is on the conservative side, i.e., tendencies toward failure should be greater in the model than in the prototype.

Test Equipment and Instrumentation

Shaking Table

The simulated earthquake was applied to the model by means of a shaking table. The top of the table was a reinforced concrete slab 8 inches thick

Quantity	Required Ratio Model : Prototype	Prototype Value	Required Model Value	Actual Model Value
Length	1 : 150	300'	2'	2'
Force	$0.87 : 150^3$	—	—	—
Time	$1 : \sqrt{150}$	—	—	—
Acceleration	1 : 1	—	—	—
Unit dry weight Model Material	0.87 : 1	110 pcf	96 pcf	96 pcf
Unit Weight Liquid	0.87 : 1	62.4 pcf	54.5 pcf	62.4 pcf
Angle of Internal Friction	1 : 1	45°	45°	42° to 48°
Cohesion of Clay	0.87 : 150	1000 psf	5.8 psf	Model E 7.2 psf Model F 7.7 psf Model G 5.0 psf
Modulus of Rigidity	0.87 : 150	12000 psi	70 psi	120 psi

which was cast within a frame of 8-in. structural steel channels. The slab was supported at each corner by a flexible steel leg which permitted the slab to move freely in the longitudinal (upstream and downstream) direction, but provided very high restraint to lateral and torsional movement. Walls 27 inches high and cross-braced to reduce local vibrations were provided to retain the model dam and reservoir. A sketch showing the general construction features of the shaking table is shown in Fig. 4. Motion was imparted to the table by a 150 pound pendulum striking against a buffer spring at the upstream end of the table. The other end of the table was anchored by a heavy spring which controlled its frequency of vibration. A general view of the shaking table with model dam and reservoir, together with some of the recording instruments, is shown in Fig. 5.

The motion produced by this type of system may be divided into two phases.(6) In the first phase, while the pendulum is in contact with the buffer spring, it acts as a system with two degrees of freedom. After the pendulum has rebounded, the ensuing table motion is a damped simple harmonic motion with the degree of damping depending upon the amount and character of the materials on the table. Typical records of the table displacements and accelerations are shown in Fig. 6.

The proximity of the vertical upstream wall of the shaking table to the face of the dam constituted a difference between model and prototype conditions which, it was felt, might cause some discrepancies in the response of the model. Specifically, it seemed that a pressure wave might be developed at this face during a downstream acceleration of the table, and that this wave might affect the model. In order to prevent this occurrence, an air cushion in the form of a blanket of inflated rubber tubes was mounted against the upstream wall. Dynamic pressure measurements indicated the effectiveness of the cushion. Prior to its installation, downstream accelerations were accompanied by pressure increases near the upstream wall. After the cushion was in place, the pressure near the upstream face decreased when the table was

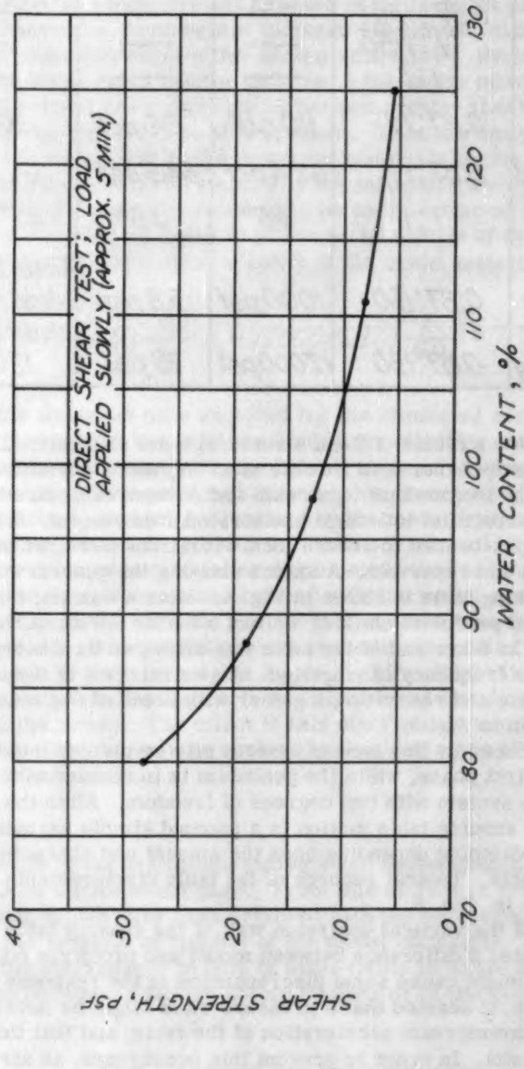


FIG. 3 - EFFECT OF WATER CONTENT ON SHEAR STRENGTH OF CLAY--
STATIC LOADING

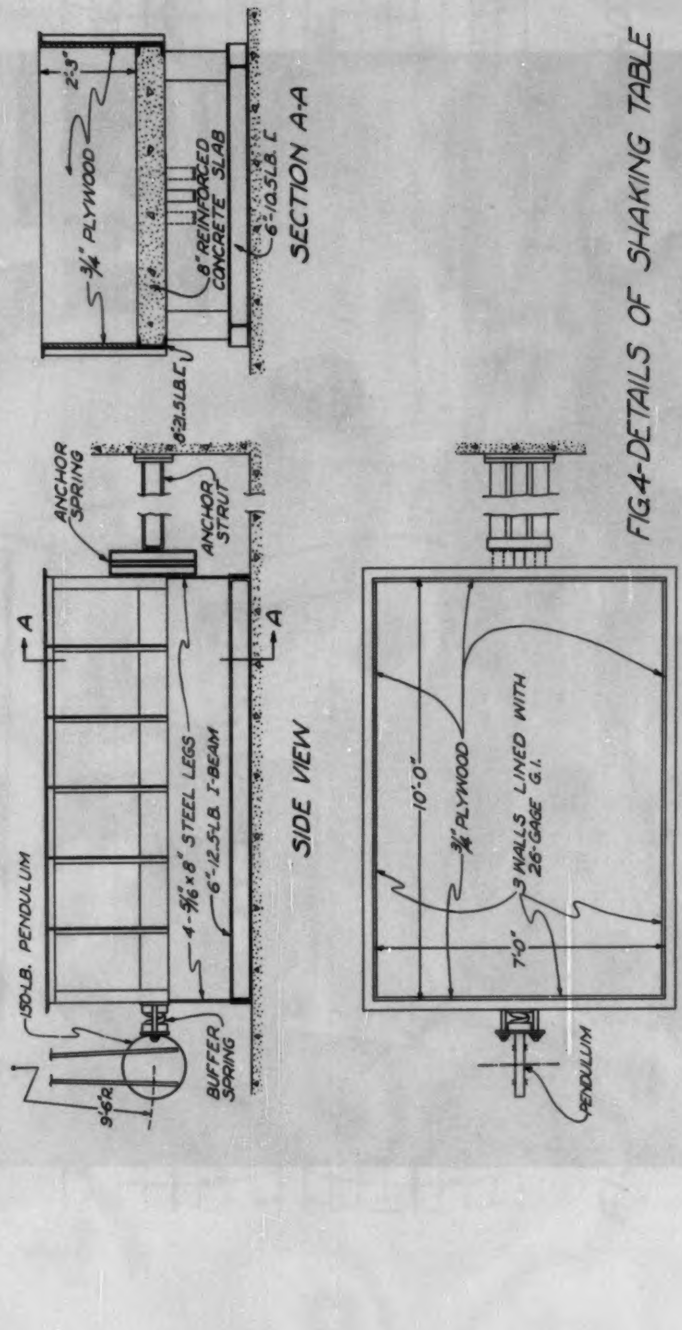


FIG 4-DETAILS OF SHAKING TABLE

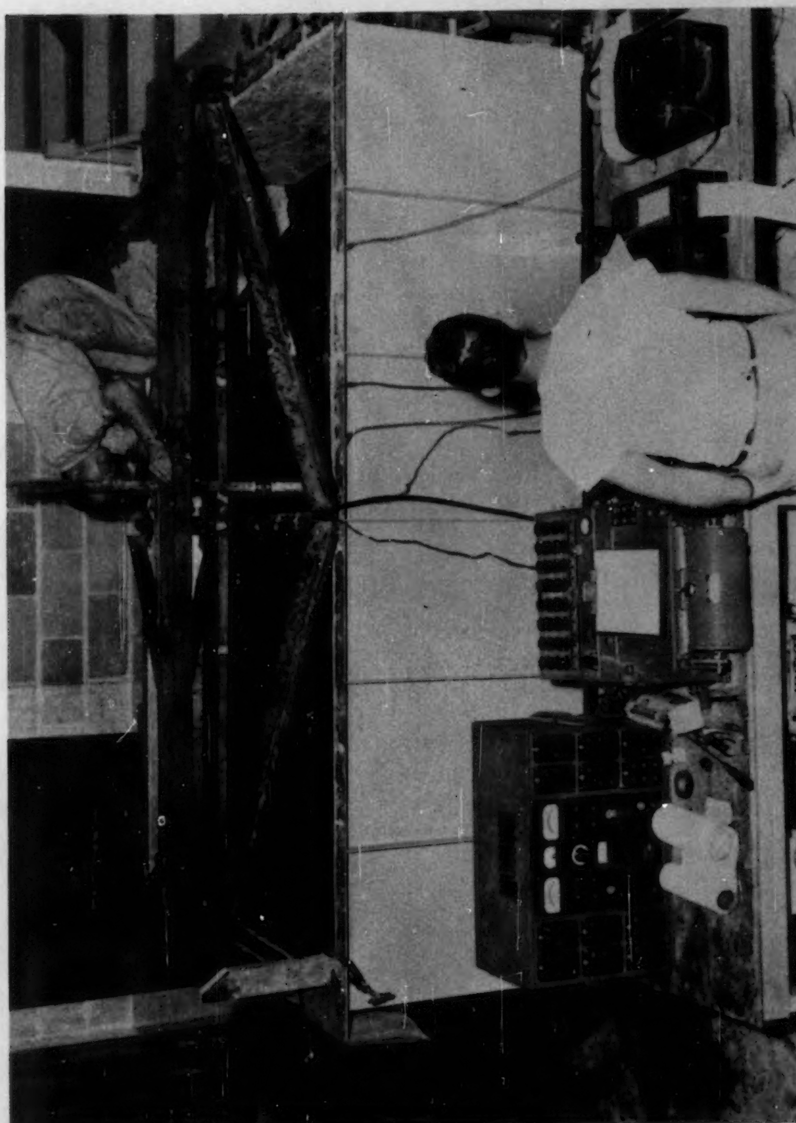


FIG.5-SHAKING TABLE AND RECORDING EQUIPMENT

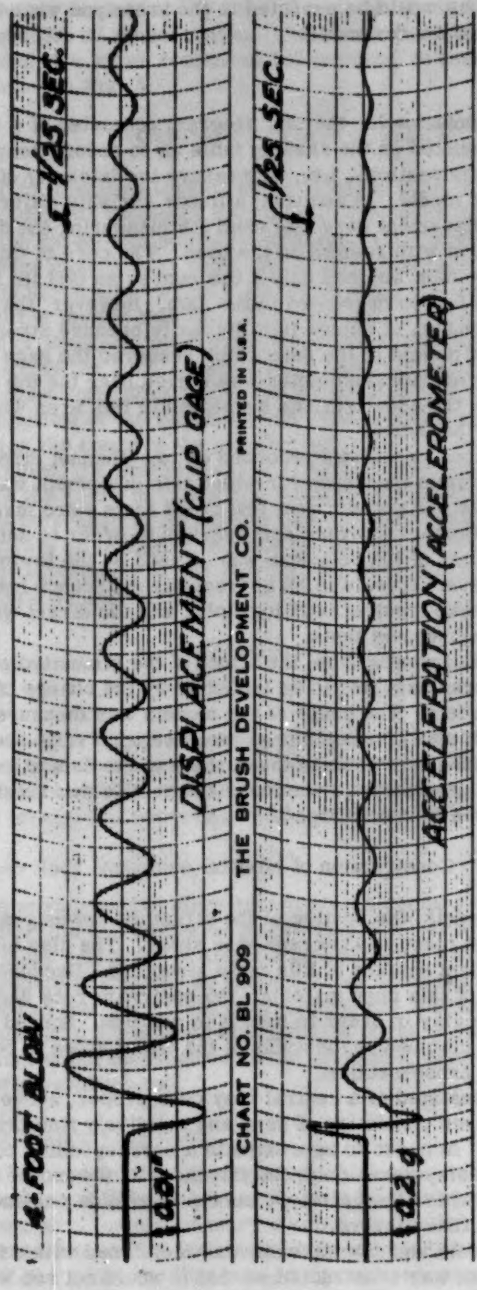


FIG. 6 - TYPICAL OSCILLOGRAMS OF DISPLACEMENT AND ACCELERATION OF SHAKING TABLE

accelerated downstream as would be expected in the prototype when the dam tended to move away from the reservoir.

Instrumentation

The primary instrumentation for the test program consisted of a clip gage and accelerometer so mounted on the shaking table as to record respectively its displacements and accelerations, i.e., to measure the intensity of the earthquake applied to the model. In addition, a linear variable differential transformer type of displacement gage was used for measuring the displacements of the top of the dam with respect to the base. The core of this gage which was attached to the dam weighed only a few ounces so that its mass had a negligible effect on the forces developed in the dam. However, the gage could be used only with quakes of rather light intensity because strong quakes were accompanied by settlement of the dam which rendered the gage inoperative. In a few tests a second accelerometer was buried near the top of the dam in order to compare the accelerations developed in this area with those developed at the base of the model.

The output of the various gages was recorded on two-channel direct writing oscillographs. The maximum frequency for which this equipment was designed is about 100 cycles per second; so a few check tests were made using a high-frequency photographic recording oscillograph in order to determine whether any significant part of the response was missed by the lower frequency recorders. However, there was no appreciable difference between the record obtained with the two types of oscillographs, and the direct writing recorder was used on all subsequent tests.

For the purpose of this investigation, the effect of the simulated earthquakes upon the models was assumed to be indicated by the change of shape or deformation of the models. The shape of the models was measured with a profileometer which consisted of a depth gage mounted upon rails above the dam. By measuring the distance down to the surface of the dam at intervals across the mid-section, an accurate measure of the profile was obtained. The profileometer may be seen in use in Fig. 5.

Construction of Models

In constructing the models, the portion of the structure downstream from the sloping core was first placed and screeded to shape. The clay blanket was then placed on the upstream surface of this main structure. Because of the very low shear strength of this core material, the upstream rock had to be built up together with the clay in order to hold it in position. Screed boards, sliding on wooden strips clamped to the walls of the table, aided in obtaining the proper shape for the cross-section.

The prototype core consisted of a central clay core proper, above and below which was a filter layer consisting of sand and relatively fine crushed rock (see Figs. 1 and 2). In order to approximate a corresponding condition in the models, which had clay cores only, the thickness of the model core was made to correspond to that for the prototype core proper plus one-half of the thickness of the adjacent filter layers.

All of the rock (upstream and downstream) was moistened with a fine spray before the core of the dam was constructed so that it would not rob water from the clay. Also, the original water content of the clay was purposely made

higher than that desired at the time of test, as it was known that some water would be lost due to consolidation. The desired shear strength of 5.8 psf. corresponded to a water content of 125 percent, in accordance with the relationship shown in Fig. 3.

General Procedure of Testing

In order to minimize loss of water from the clay blanket due to consolidation, the model was tested as soon as possible after its construction was completed. Each model was subjected to a series of "quakes" of increasing magnitude. These quakes each consisted of an initial impact of the pendulum plus two rebound blows after which the shaking table was permitted to oscillate freely until the motion was finally damped out. During each motion, records were taken of acceleration and displacement of the shaking table (base of dam) and of the top of the dam. Also, profile measurements were made prior to filling the reservoir, after filling the reservoir, and after each test quake.

The magnitude of the accelerations developed by the quakes was controlled by specifying the chord of the arc through which the pendulum swung. For the weaker motions, the magnitude of the maximum acceleration was nearly proportional to the chord distance; for the stronger motions, the maximum accelerations increased more rapidly than the chord distances.

Immediately after testing the model, the reservoir was drained and six representative samples of the clay blanket were taken so that its shear strength could be determined.

Results of Tests

General

A total of eight different models was tested. Two of these consisted of the downstream rock structure for a sloping core dam (i.e., without any clay core), five were models with sloping cores and having water in the reservoir at various levels, and one was a model of a dam with a central clay core.

Downstream Rock Structure Only

The two models on the downstream rock structure only for a sloping core dam (downstream and upstream slopes of 1.40:1) were conducted to study the change in slope and crest settlement of a rock fill placed approximately at its angle of response (1.35:1). At accelerations up to 0.4 g the profiles for both models showed that very little change occurred and the settlement at the crest amounted to 0.6 percent of the height of the model. This settlement is no more than would be expected in the prototype, under static conditions, over a period of several years.

Model with Sloping Clay Core

Of the five models with sloping core which were tested, only the results of the third and fourth (Models E and F) will be discussed. The first two models were used for preliminary studies, to develop methods of construction and instrumentation. The last model was used for check purposes.

Model E had a sloping clay core, a downstream slope of 1.40:1, and an upstream slope of 2.25:1, as shown in Fig. 1. The model was tested at 3 hours

after construction with water at the full reservoir level. The average water content of the core immediately after test was 115 percent; and the corresponding shear strength was 7.2 psf (Fig. 3). The observed accelerations and displacements are given in Table 1 and the profiles in Fig. 7. It should be noted that these profiles indicate accumulated displacements. The effect of any one quake is indicated by the change in profile from one curve to the next.

The first profile in Fig. 7 shows a considerable change in section due to filling the reservoir to its high-water elevation (1.82 ft.). The crest settled 0.2 percent of the height of the model, and moved downstream about 2.5 percent of the height of the model. Thereafter little change occurred until an acceleration of 0.4 g was reached; at that acceleration the crest settled noticeably and rounded off, the upstream face moved downstream, and the downstream face bulged. At higher acceleration these movements continued.

In the upstream face, the principal changes in section occurred in the upper half, even at the higher accelerations. From mid-height to three-quarter-height, the slope flattened considerably; from three-quarter-height to the rounded crest, the slope remained constant but this portion of the face shifted downstream.

The bulging of the downstream face progressed as follows: At an acceleration of 0.4 g, the slope of the upper half flattened somewhat; and at 1.1 g the slope of the upper half was becoming still flatter, and the slope near the base was becoming steeper.

Model F was identical with Model E, but was tested under different conditions of water load. The model was tested 2 hours after construction with the reservoir empty at accelerations up to 0.36 g, and then was tested again with the reservoir 0.4 full. The average water content of the core, determined immediately after test, was 113 percent; and the corresponding shear strength was 7.7 psf. The accelerations and displacements are given in Table 1, and the profiles in Fig. 8.

With the reservoir empty and the model subjected to accelerations up to 0.36 g, the top half of the model settled without appreciable change in slopes, and the lower half of the upstream face bulged slightly; these movements indicated that some slight slippage had occurred in the clay core. A stability analysis (not including earthquake effects) indicated that the clay core and upstream rock were less stable when the reservoir was empty than when it was full.

With the reservoir filled to a depth of 0.80 ft., which had been determined by stability analysis (not including earthquake effects) to be the critical depth of reservoir producing the least stable condition, a new series of tests was made. The results are also shown in Table 1 and Fig. 8.

At an acceleration of 0.3 g the crest settled an additional 0.8 percent of the height of the model, the upper half of the upstream face settled slightly, and the upstream face bulged slightly near the base. There was no change in the downstream face at this acceleration. At higher accelerations the motions of the upstream face continued to a marked extent, and indicated a slippage in the clay core.

Model with Center Clay Core

Model G was built with a vertical clay core at the center of the section, as shown in Fig. 2. The shape of the prototype was similar to that of Mud Mountain Dam, but the rock and clay used in Model G were the same as those

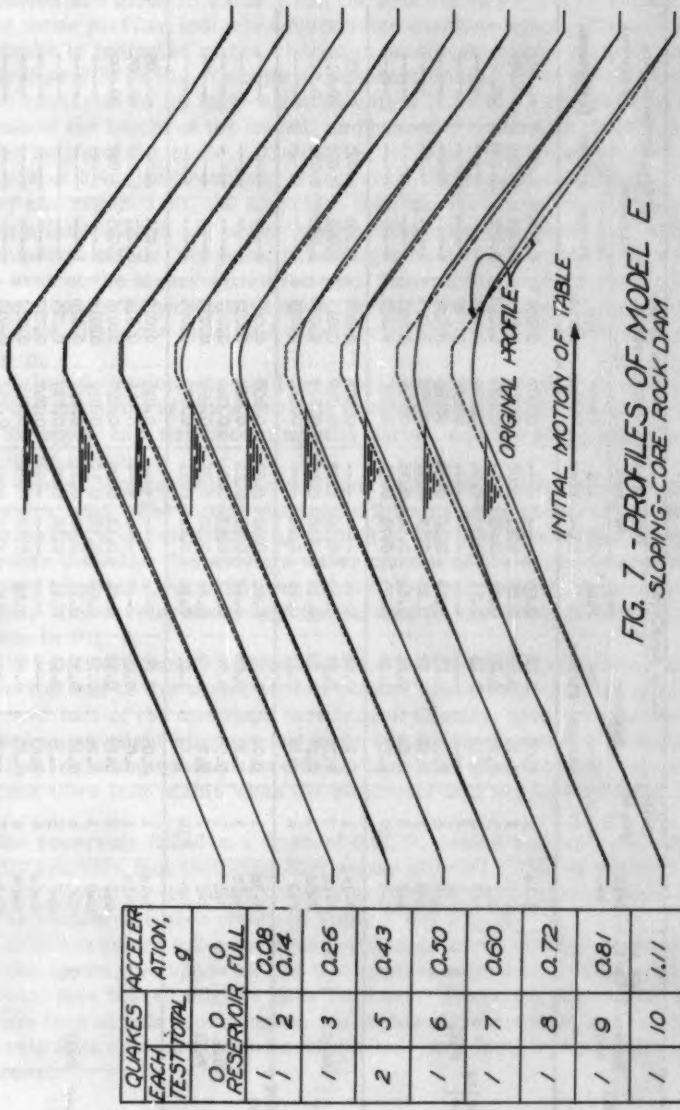
TABLE 1. -- RESULTS OF TESTS ON MODELS E, F, and G

Models E & F: Sloping core; downstream slope of face 1.40:1; upstream slope of face 2.25:1

Model G: Center core; lower portion of downstream and upstream slopes 2.23:1 and upper portion 1.75:1

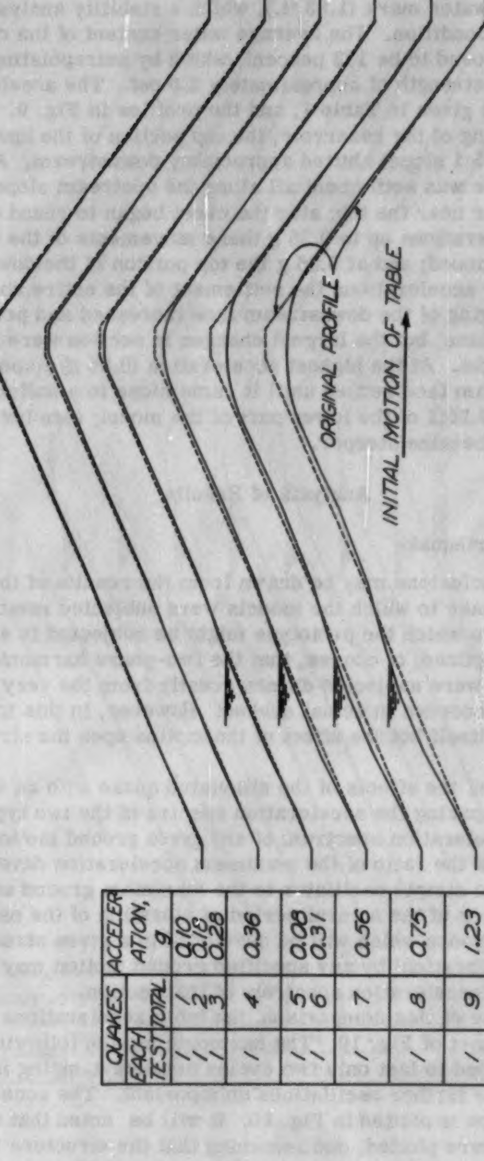
Model	Reservoir	Age at Test, hr.	Test No.	Chord Length of Blow, ft.	Table Acceleration, g					Maximum Table Displacement, in.		Top Displacement, in.		
					Maximum		Avg. Peak[a]	Downstream Maximum				Maximum		Permanent
					Downstream	Upstream		2nd Blow	3rd Blow	Downstream	Upstream	Downstream	Upstream	Downstream
E (Sloping Core)	Full	3	E-1a	1	0.08	0.08	0.05	----	----	0.007	0.006	0.006	0.004	+0.001
			E-1b	2	0.14	0.12	0.06	0.11	0.08	0.013	0.012	0.022	0.010	+0.003
			E-2	3	0.26	0.20	0.09	0.21	0.13	0.020	0.018	0.055	0.020	-0.050
			E-3	4	0.43	0.41	0.13	0.32	0.24	0.029	0.023	-----	-----	-----
			E-4	5	----	----	----	----	----	0.033	0.029	-----	-----	-----
			E-5	6	0.60	0.54	0.18	0.40	0.30	0.043	0.032	-----	-----	-----
			E-6	7	0.72	0.72	0.20	0.49	0.32	0.051	0.038	-----	-----	-----
			E-7	8	0.81	0.84	0.24	0.50	0.36	0.056	0.041	-----	-----	-----
			E-8	10	1.11	1.28	0.33	0.78	0.53	0.075	0.052	-----	-----	-----
F (Sloping Core)	Empty	1-1/2	F-1a	1	0.10	0.06	----	----	----	0.007	0.007	0.001	0.001	+0.000
			F-1b	2	0.18	0.12	0.09	0.10	0.09	0.015	0.015	0.024	0.004	+0.013
			F-1c	3	0.28	0.20	0.12	0.18	0.15	0.022	0.020	0.050	0.032	-0.011
			F-2	4	0.36	0.32	0.13	0.20	----	0.030	0.025	-----	-----	-----
	0.4 Full	2	F-3a	1	0.08	0.07	0.04	0.02	0.02	0.007	0.007	-----	-----	-----
			F-3b	3	0.31	0.28	0.12	0.21	0.14	0.020	0.017	-----	-----	-----
			F-4	5	0.56	0.50	0.17	0.42	0.24	0.040	0.027	-----	-----	-----
			F-5	7	0.75	0.70	0.24	0.60	0.35	0.055	0.035	-----	-----	-----
			F-6	10	1.23	1.20	0.36	0.81	0.61	0.075	0.055	-----	-----	-----
G (Center Core)	Full	3	G-1a	1	0.05	0.05	----	----	----	0.015	0.010	0.011	0.002	+0.005
			G-1b	1	0.05	0.05	----	----	----	0.015	0.010	0.012	0.003	+0.005
			G-2	2	0.12	0.12	0.05	0.11	0.06	0.015	0.015	0.015	0.007	+0.000
			G-3	3	0.26	0.23	0.08	0.18	0.12	0.022	0.015	-----	-----	-----
			G-4	4	0.37	0.36	0.11	0.30	0.21	0.030	0.020	-----	-----	-----
			G-5	5	0.51	0.51	0.14	0.40	0.26	0.037	0.025	-----	-----	-----
			G-6a	6	0.63	----	----	----	----	0.042	0.032	-----	-----	-----
			G-6b	6	0.65	0.66	0.18	0.47	0.34	0.042	0.032	-----	-----	-----
			G-7	7	----	----	----	----	----	0.052	0.037	-----	-----	-----
			G-8	8	0.95	0.95	0.26	0.70	0.50	0.060	0.042	-----	-----	-----

[a] Average of maximum second and third downstream acceleration and second upstream acceleration, as an indication of harmonic accelerations.



QUAKES EACH TEST	QUAKES TOTAL	ACCELERATION, g
0 RESERVOIR	0	0 FULL
1 1	1 2	0.08 0.14
1	3	0.26
2	5	0.43
1	6	0.50
1	7	0.60
1	8	0.72
1	9	0.81
1	10	1.11

FIG. 7 - PROFILES OF MODEL E
SLOPING-CORE ROCK DAM



QUAKES		ACCELERATION, g
EACH TEST	TOTAL	
1	1	0.10
1	2	0.18
1	3	0.28
1	4	0.36
1	5	0.08
1	6	0.31
1	7	0.56
1	8	0.75
1	9	1.23

FIG. 8 - PROFILES OF MODEL F
SLOPING-CORE ROCK DAM

used in the sloping-core models. Model G had upstream and downstream slopes of 1.75:1 over the upper 0.37 of its height, and slopes of 2.25:1 over the lower 0.63. It was tested at 3 hours after construction, with the reservoir filled to the high-water mark (1.83 ft.), which a stability analysis had indicated to be the critical condition. The average water content of the core immediately after test was found to be 132 percent, which by extrapolating from Fig. 3 indicated a shear strength of approximately 5.0 psf. The accelerations and displacements are given in Table 1, and the profiles in Fig. 9.

During the filling of the reservoir, the top portion of the upstream face (the portion of 1.75:1 slope) shifted appreciably downstream. At an acceleration of 0.05 g there was settlement all along the upstream slopes, the settlement being greater near the top; also the crest began to round off and dip upstream. At accelerations up to 0.26 g these movements of the upstream face and the crest continued; and at 0.26 g the top portion of the downstream face bulged. At higher accelerations the settlement of the entire upstream face continued and bulging of the downstream face increased and progressed downward to near the base, but the largest changes in section were still in the top portion of the model. At the highest acceleration (0.95 g) imposed on this model, the upstream face settled until it came close to a uniform slope flatter than the original 2.25:1 of the lower part of the model; also both slopes of the downstream face became steeper.

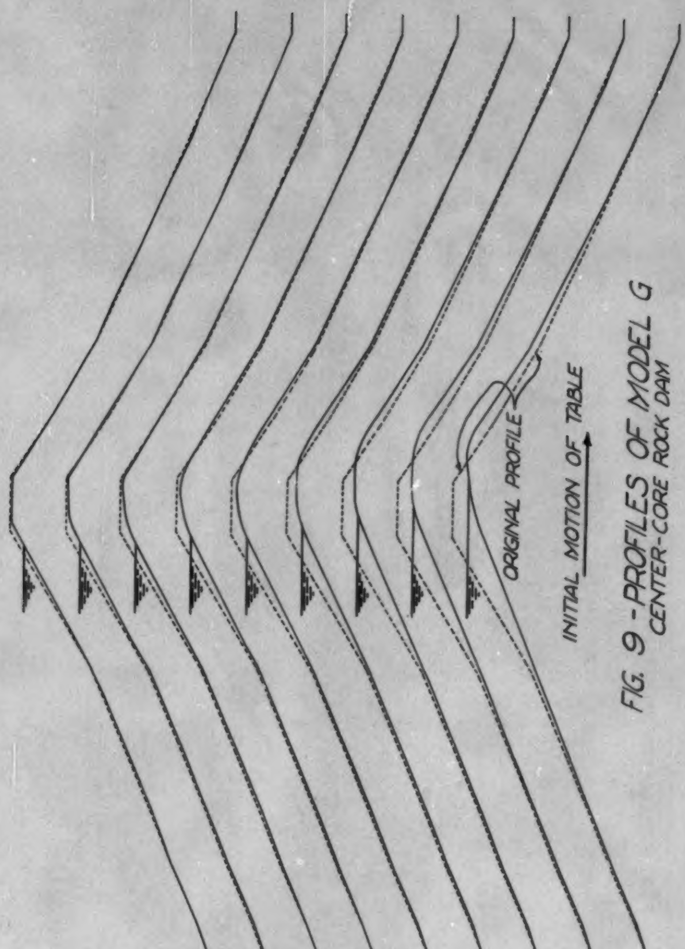
Analysis of Results

The Simulated Earthquake

Before any conclusions may be drawn from the results of these tests, the simulated earthquake to which the models were subjected must be compared with the motions to which the prototype might be subjected in an actual earthquake. It is recognized, of course, that the two-phase harmonic motion to which the models were subjected differs greatly from the very irregular and erratic motions recorded in actual quakes. However, in this investigation it is not the motion itself but the effect of the motion upon the structures which is of importance.

A comparison of the effects of the simulated quake with an actual one is best made by comparing the acceleration spectra of the two types of motions.(7) The acceleration spectrum of any given ground motion may be defined as a graph of the ratio of the maximum acceleration developed by the ground motion in a simple oscillator to the maximum ground acceleration, plotted as a function of the natural period of vibration of the oscillator. Thus, the maximum response which will be developed in a given structure (having a known period of vibration) by any specified ground motion may be obtained directly from the acceleration spectrum of that motion.

For the purpose of this comparison, the table accelerations were idealized as shown in the inset of Fig. 10. The harmonic motion following the initial impact was assumed to last only two cycles because damping in the structure tended to make the further oscillations unimportant. The acceleration spectrum of this motion is plotted in Fig. 10. It will be noted that two separate spectral curves were plotted, one assuming that the structure was undamped, and the second assuming a viscous damping factor of 20 percent of critical damping. For comparison, two spectral curves for the North-South component of the El Centro earthquake of May 18, 1940 (taken from Reference 7,



ACCELERATION, g	IMPACTS TOTAL	IMPACTS EACH TEST
0 RESERVOIR FULL	0	0
0.05	2	2
0.12	3	1
0.26	4	1
0.37	5	1
0.51	6	1
0.64	8	2
0.79	9	1
0.95	10	1

FIG. 9 - PROFILES OF MODEL G
CENTER-CORE ROCK DAM

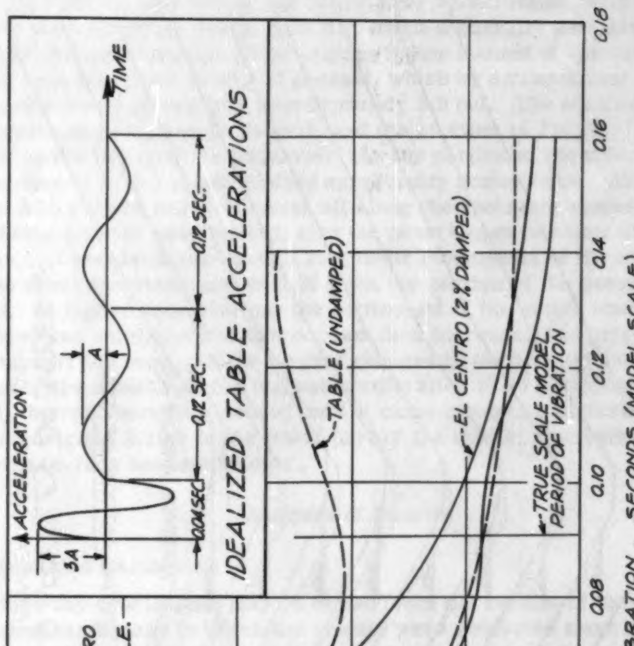


FIG.10- COMPARISON OF ACCELERATION SPECTRA

p. 769) have been smoothed out slightly and plotted on the same graphs. The time scale of the El Centro earthquake spectrum has been reduced to the model scale in order to make the comparison.

The agreement between the two sets of curves is quite striking. Comparing first the undamped table motion spectrum with the earthquake spectrum for 2 percent critical damping, it is seen that the two are nearly identical for structures having periods between 0.03 and 0.07 seconds (model scale). Structures with longer periods, however, tend to develop resonance with the low frequency harmonic motion of the table, and consequently the table spectrum increases in amplitude again for these longer periods in contrast to the earthquake spectrum which shows a continual decrease toward the longer periods.

Comparison of the spectral response curves for structures having 20 percent of critical damping is of greater significance, because the rock-fill dam is a structure having a large capacity for internal damping. These curves are practically identical throughout the entire range of periods considered, leading to the conclusion that the maximum response developed in the dam by the simulated earthquake is very nearly the same as the maximum response which would be developed by an actual earthquake.

The effect of the lack of similitude in the shear modulus of the model material is also indicated by the spectral response curves. The principal effect of this discrepancy was to reduce the natural period of vibration of the models from 0.091 seconds to 0.069 seconds. The acceleration spectra show that while the spectral response ratio would be only 0.80 if the model had the correct rigidity, the more rigid material which was actually used had the effect of increasing this ratio to about 1.20. Thus the lack of similitude in this respect clearly increases the forces developed in the model.

Dynamic Response of the Models

The response of the model to the simulated earthquake was evaluated from the records of the displacement of the top of the dam relative to the base. These displacements together with the records of the table displacement made it possible to deduce the motion of the structure as a whole. The typical motion observed is shown in Fig. 11. The upper curve shows the total movement at the gage position near the top of the dam, the lower curve shows the table motion, i.e., the movement of the base of the dam. The position of the axis of the dam at intervals during the motion is shown in the diagrams between the two curves.

These curves and diagrams show clearly that the base of the dam first moved downstream, leaving the top almost stationary. Then, after a time lag of 0.02 to 0.03 seconds, the top of the dam whipped over beyond the base. In the following cycle or two, the top quickly came into phase with the base; and during the ensuing harmonic motion the whole model moved as a unit.

In general, as a result of distortions within the dam the top moved farther than the base during the harmonic oscillation and therefore was subjected to higher accelerations. This amplification amounted to 25 to 35 percent in the tests which were recorded; however, it had but little significance because the maximum accelerations were associated with the initial blow rather than with the harmonic motion. The initial shock was felt more strongly at the base than at the top, the flexibility of the model serving as a sort of shock absorber which prevented transmission of the maximum accelerations directly to the top.

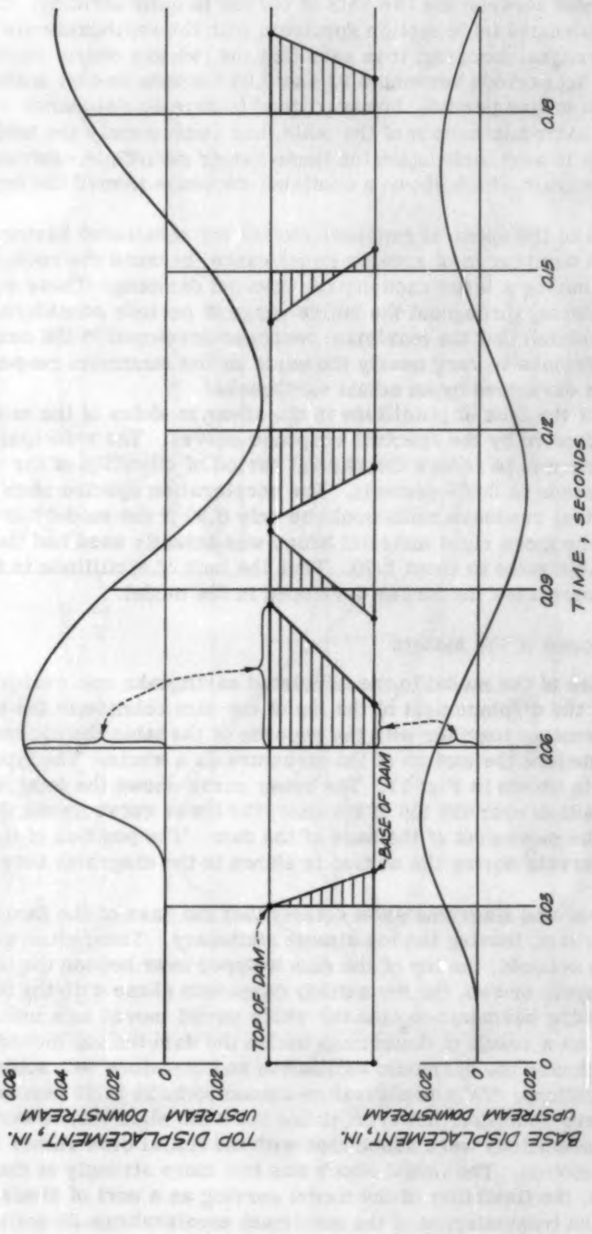


FIG.11-DISPLACEMENT OF TOP AND BASE OF MODEL E (0.4 FULL)

CONCLUSIONS

The following conclusions regarding the results of this investigation seem to be justified:

1) The strength of the model was designed in proper scale to the prototype structure, and the acceleration spectrum indicates the model quake to have been approximately equivalent to a typical prototype earthquake. Therefore, it may be concluded that the results observed in the model may be extrapolated to the prototype; i.e., the effect of a test quake on the model should be equivalent to the effect of an actual earthquake (having the same maximum acceleration) on the prototype.

2) The applied quakes produced no significant effects (appreciable changes of section) on the models until the progressively increasing accelerations exceeded 0.4 g. The strongest accelerations which have been recorded with reliable strong-motion seismographs during actual quakes have been less than 0.4 g. Therefore it may be concluded that rock-fill dams similar to these models would not be seriously damaged by earthquakes of the magnitudes observed in the Western States during the last 30 years.

3) Even when the accelerations of the test quakes were increased to more than 1 g, the models suffered only minor changes of shape, indicating that with the strongest conceivable quakes the prototype structures would show damage only in the attached rigid structures and appurtenances. It is unlikely that settlements large enough to cause overtopping of the structure would result.

4) The high degree of earthquake resistance exhibited by these models may be attributed to two principal factors. First, the materials of which the dam is constructed, both clay and granular, show increased strength when subjected to dynamic loading (Refs. 4 and 8); hence the capacity to resist earth quake forces is greater than the capacity to resist static forces. Second, the rock-fill dam is naturally a very flexible structure and can undergo large distortions without appreciable damage.

In connection with this latter point, it may be noted that the customary method of designing structures to resist earthquakes is not applicable to rock-fill dams. The usual method of taking account of earthquake effects in design is to assume a reasonable value for the maximum ground accelerations, say 0.1 g, and then to apply a static horizontal force to the structure equal to the mass of the structure multiplied by this acceleration. This equivalent static force approximates the inertia forces to which the structure would be subjected if it were completely rigid. However, in a rock-fill dam, this force has no significance, for if it should exceed the shearing resistance of the dam, the only effect would be to initiate shearing displacements. Before much displacement could occur, the direction of the ground acceleration would be reversed as the ground oscillated back, and the shearing displacements would cease. Because of the flexible nature of the dam structure, these shearing displacements would be of no importance. The action would be similar to the response of a deck of playing cards to a small horizontal oscillation of the table on which it lay. So long as the displacement of the base was only a small percentage of the width of the deck, no harm would result. As noted above, settlements and cracking of the attached rigid structures would occur, but the basic dam structure would be undamaged.

5) These conclusions apply to both types of rock-fill dams with impervious cores. However, the results of this investigation indicate that the sloping-core

type is somewhat more earthquake resistant than the center-core type, because its structure is more closely bound together. In the center core dam, the vertical core breaks the continuity between the upstream and downstream portions of the rock fill and constitutes a zone of weakness, while the entire structure of the sloping-core dam acts as a unit. Either type of dam can be made stable and will exhibit high resistance to earthquake action; but, because of its greater rigidity, the sloping core dam should show less of the settlement which accompanies the shearing distortions.

ACKNOWLEDGMENTS

The work described in this paper was sponsored by the International Engineering Company of San Francisco, California. Messrs. D. J. Bleifuss and J. P. Hawke, of the International Engineering Company, gave much valuable advice and assistance. The investigation was performed under the immediate supervision of Professor R. E. Davis, of the University of California. Mr. George Wendell, Associate Research Engineer, worked with the authors on all phases of the project.

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JOURNAL

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 942)

Discussion of
"FOUNDATION TREATMENT FOR EARTH DAMS ON ROCK"

by Thomas F. Thompson
(Proc. Paper 548)

THOMAS F. THOMPSON,[1] AFF. ASCE.—Discussion by Mr. Thomas W. Fluhr. Mr. Fluhr's discussion further emphasizes some of the important problems to be contended with in the design and construction of earth dams, and particularly those which are related to securing the best practicable foundation for these structures, even though conditions as provided by nature may be far from optimum. He points out that a conventional or stereotyped program of pressure grouting alone, no matter how well planned and carried into effect, will not necessarily remedy all the defects or geological shortcomings which may be inherent to a construction site. Rather, each location requires individual assessment of its adverse properties, both at the foundation surface or below, which will require the taking of corrective measures, such assessment not being confined merely to planning and design stages, but assuming even a greater importance during the construction phase, at which time the "chips are down," and it is imperative that a sound and rapid decision be made as to the proper means of contending with minor, but never-the-less important geological features that are disclosed by stripping. It is the foundation engineer's responsibility to come up with the best means of arriving at full insurance against damage to the structure because of the nature of the foundation. Even though pressure may be strong for getting the fill under way because of restricted construction schedules, there should be no temporization with regard to carrying out recommended treatment, when recommendations are based on a thorough appraisal of local geological conditions, experience, and mature judgment.

Discussion by Mr. Horace A. Johnson

The writer does not agree with the opinion expressed by Mr. Horace A. Johnson "the only way that damage can occur at the foundation is for there to be open passages at the rock surface which extend practically the full width of the impervious section of the dam." In the case of a foundation that is composed of blocky jointed rock, with evident separation between the blocks (a condition that is not uncommon at the surface of granitic foundations), there is ample opportunity for the creation of hydraulic velocities within joints or fractures subjected to reservoir head which are capable of eroding fine grained materials of low cohesion. This would be true even though no particular joint system had great linear continuity. When abutments are steep, as is commonly the case at mountainous or foothill dam sites, the direction of flow of seepage waters at or near foundation level, which manage to bypass the grout curtain, is not necessarily towards the blanket drain illustrated on Mr.

1. Resident Engr., The Ralph M. Parsons Co., New Delhi, India.

Johnson's Figure 1, but more often is in a direction which is nearly parallel or at an acute angle to the axis of the dam and reaches the buried valley floor upstream from the blanket, in which case beneficial aspects of the blanket as a measure for stopping the movement of fines eroded from the base of the fill are considerably lessened.

Mr. Johnson cites Isabella dam, which is located upon a granite foundation, as an example where, in his opinion, surface foundation treatment was of questionable value, and describes the comparatively conservative design of the dam, bringing forth the slight difference in composition of the decomposed granite which comprises Zones 1 and 2 of its section. This material has little cohesion, and although it forms an excellent, relatively impervious type of rolled fill, it is not characterized by high resistance to erosion, as is the case with materials of more plastic nature. Mr. Johnson further implies that the fractures in the rock at this site do not have continuity. The writer disagrees with this concept, since foundation mapping of the stripped rock surface in the river bottom section, which was the only area cleaned to the extent as would permit tracing fractures for long distances, disclosed individual or interconnected systems of joints and minor shears with trend roughly parallel to the strike of the Kern fault, some 2500 feet to the east, which could be traced for nearly the entire base-width of the structure. The fact that the joints in the abutments could be seen only in the core trench does not mean that they do not exist beyond, but only that the rock was not exposed elsewhere to the extent that joints could be mapped. Where joints crossed the core trench, as a conservative measure in keeping with other provisions of design, it was decided to provide a surface seal of all fractures not filled by foundation grouting and into which the tamping of the initial lifts of earth fill would be difficult, to eliminate the possibility of erosion taking place at the base of the fill, since the added security gained thereby was of insignificant cost.

Regarding the dam failure about which Mr. Johnson inquires, the features of which are illustrated on Plates I, II and III of the paper, there is little room for doubt that the immediate cause was inadequate treatment of the foundation, since no grouting of the foundation was performed during construction, but was being done as an adjunct to reconstruction, with considerable grout "take." The dam failed the first time that the reservoir was filled to spillway level. As shown on Plate II, a fissure had developed in the fill a few inches above the foundation, in an area where several small springs were present (Plate III). These flows, with connection to the reservoir by way of open joints, fed water directly into the fissured base of the fill, with inevitable consequences.

Discussion by Mr. Giovanni Rodio

Mr. Giovanni Rodio proposes an interesting method for grouting through the use of a gallery located in the foundation. To the writer's knowledge, this has never been done in the United States on an earth dam project. He lists certain undeniable advantages forthcoming from use of a grout and drainage gallery which are not gained when the grouting is from the bottom of the core trench or from the crest of the completed embankment, but admits that the cost of the method proposed would be high.

Although the use of a gallery might be justified in an extreme case, as is typified by the lower of the two sections on Mr. Rodio's Figure 1, it is doubtful that this scheme will find broad acceptance for the following reasons:

1) The gallery roof would require costly reinforcing to enable it to withstand earth pressures.

2) Its construction would delay the start of fill operations, a feature which would be of importance if the structure was to be completed in one season, as are many moderate sized dams in the western United States.

3) If the foundation is badly broken or comparatively soft, the introduction of a rib of hard material, as would be the case with the gallery structure, could set up conditions conducive to cracking of the fill.

4) The system of foundation drains discharging into the gallery would require use of a sump pump to keep the gallery unwatered. Tailwater leakage through a relatively open type of formation might make pumping very costly.

Discussion of
"SEEPAGE FORCES IN A GRAVITY DAM BY ELECTRICAL ANALOGY"

by Horace A. Johnson
(Proc. Paper 757)

SERGE LELIAVSKY[1] M. ASCE.—It should be emphasized that the problem treated in this paper (the potential flow theory as applied to the calculation of masonry dams) has been investigated before. However, the author's lack of information may partly be excused because the report of the ASCE Subcommittee on Uplift in Masonry Dams (Paper No. 2531, "Transactions," ASCE, Vol. 117, 1952) failed to recognize the importance of the application of the theory of potential flow to the calculation of the uplift in dams.

The engineering profession should realize that uplift is a physical theory, and as such, calls for a mathematical solution. It cannot be solved by "hunches," etc.

In explaining the advance achieved in his paper the author states (see "Synopsis"):—

> "Flow is assumed to occur through both the concrete and the rock foundation, which is contrary to the usual assumption of no flow in concrete or rock."

This statement is contrary to facts. Such archaic assumptions are not, and have never been, usual. As early as 1928, Hoffman gave the flow-net diagram shown in Fig. 1, according to which the flow occurs through both the concrete of the dam, and also through the rock of the foundation. In 1947 the writer developed Hoffman's method and gave the pressure lines and the volumetric loading system, again, for seepage occurring concurrently in the dam and in its foundation.[2] In 1934 Prof. Terzaghi investigated the phenomenon of different values of the permeability coefficient for the concrete of the dam, and for the rock of the foundation. He gave flow-net diagrams, including stream lines and curves of equal potential, for the following cases:—

1) Equal permeabilities of dam-concrete and foundation rock, or (in the notation adopted by the author):

$$K_f = K_c$$

2) Both dam and foundation permeable, but the permeability of the foundation decreases from upstream to downstream.

3) Same as Case 2, but with inverse order of variation of permeability.

4) Pores on the downstream face of dam sealed by frost action.

5) Evaporation discharge on the downstream face of the dam effective.

1. Civ. and Hydr. Engr., Cairo, Egypt.
2. S. Leliavsky Bey, "Sazillism: Its Origin and Evolution," The Engineer, London, May, 1947.

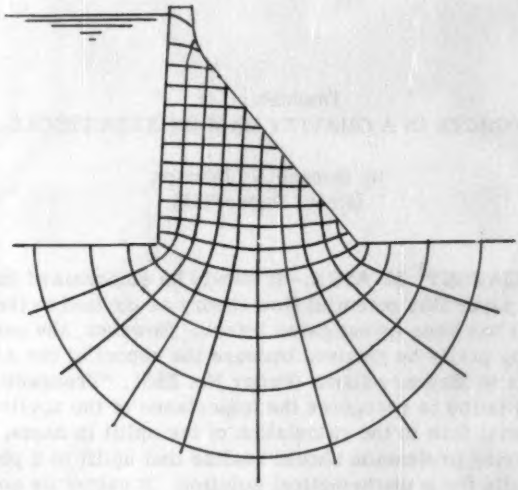

Figure 1.

6) Open cracks.

These results, as well as many other contributions to the theory of potential flow in dams, need to be investigated before deriving conclusions from, or designing the arrangement of, the author's tests.

The author's discussion of his test results may create the false impression that they are, in some way or another, correlated with Mr. Keener's diagram. This, of course, is entirely wrong. In the first place, the information brought to light in Mr. Keener's extremely valuable and authoritative paper, was rather of a negative character.[3] It showed that in spite of the almost unlimited resources of the Bureau of Reclamation, no new original material was capable of being produced after twenty years of observation, so long as the old traditional auscultation methods were employed. For instance, the pattern of Mr. Keener's diagram reproduced by the author, does not differ in any essential from that which Ivan E. Houk published some twenty years earlier.[4]

In the second place, it is the author's own results which, if contemplated in their historic perspective, will be found to contain no material capable of being converted into general conclusions. If compared with the earlier flow-net diagrams, the fact that the foundation potential falls to 15% of the total, will obviously be found to be the result of the exaggerated effect of curtains and drains, and can by no means be attributed to the assumed permeability of the rock foundation—an obvious conclusion, showing that the similarity of the author's and Mr. Keener's curves is purely coincidental. It would be quite wrong, therefore, to say that the former confirm the latter, or vice versa.

With the abundance of research available in technical history, there was no need for the author to conduct 58 groups of tests to demonstrate the efficacy of the drains. This device was first introduced by the greatest

3. S. Leliavsky Bey, Discussion of "Uplift Pressure in Concrete Dams" by Kenneth B. Keener, Trans. ASCE., Vol. 116, p. 1247.
4. "Civil Engineering," 1933, September.

dam-builder of all times, Intze, and, for some time, was a characteristic feature of all his designs. Subsequently it was relegated to the group of secondary measures, because of the observed vagaries of the discharge yielded by individual drains. The point of the controversy centres, therefore, on the physical discharge capacity of each drain, and the author has contributed no information throwing new light on this point. Although the experimentation technique is admirable, the writer is completely at variance with the author as to the logic of the author's program and the interpretation of his tests.

[illegible] of all three, blue, and, the [illegible], [illegible] factors of all the designs. Subsequently, it was [illegible] to the [illegible] [illegible], evidence of the observed [illegible] in the [illegible] [illegible] by [illegible] design. The point of the [illegible] [illegible], on the [illegible] discharge capacity of [illegible], and the [illegible] [illegible] ed to [illegible] throwing new light on this point. Although [illegible] this technique is [illegible], the writer is [illegible] of [illegible] with the [illegible] on the logic of the [illegible] and the interpretation of the [illegible].

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DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

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OCTOBER, 1955-OCTOBER, 1956

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CURRENT RESEARCH AND DEVELOPMENT

Like Bertha, the Sewing Machine Girl, we have been rescued once more in the nick of time from a fate worse than death: Mr. R. F. Legget has just sent us a detailed account of the history and present status of the Associate Committee on Soil and Snow Mechanics, of the National Research Council of Canada; and Professor G. F. Sowers has contributed an article on soils activities at the Georgia Institute of Technology. Not only does this generous bequest render us materially solvent for the current issue of the Newsletter, but the extent of the contribution from Mr. Legget warrants its publication in two parts, and leaves us with a head start for the July number. Don't miss the next exciting installment to find out how it all comes out.

Associate Committee on Soil and Snow Mechanics, National Research Council, Canada

The Associate Committee on Soil and Snow Mechanics is one of about thirty technical committees which assist the National Research Council in the discharge of its responsibilities in connection with scientific and industrial research in Canada. Each of the Associate Committees is given responsibility for a special field of scientific interest. They are appointed by the Council and report to the Council, being made up of individuals selected for their special competence and interest in the fields concerned, usually appointed for a three-year term of office. None of the committees includes any officially designated representatives of any organization although the selection of individual membership does link the Council, informally, with most of the organizations in Canada concerned with research in the designated fields.

The Associate Committee on Soil and Snow Mechanics has as its primary concern the study of the fundamental physical, chemical and mechanical properties of soil, with special reference to the trafficability of vehicles and, as time permits, in a general way with reference to other practical problems, of both a military and civilian character. Since the surface of Canada, unlike that of other countries, includes large areas of muskeg, and of permanently frozen ground, and is for long periods almost wholly covered with snow and ice, the Committee includes within its field the study of the corresponding properties of muskeg, permafrost and of snow and ice. Within the field so defined, the Committee gathers the information required to enable it to be fully informed as to the state of knowledge in the field, and particularly research work in progress in Canada, and advises the Council on the collection and dissemination of research information generally in this field. The Committee is to investigate and stimulate research work in Canada in its assigned field and serves in a liaison capacity between the Council and Government, educational and other organizations or agencies, engaged in or concerned with research in its assigned field.

Brief History. The Committee was organized in 1945 and held its first meeting on April 20 of that year. It was set up to deal with a specific urgent military problem related to soils which was recognized only in the closing

stages of the Second World War. Initially, the Committee was a small one consisting of equal numbers of civilian and military members. It functioned effectively and developed close liaison with similar groups working on the same problem in the United States and Great Britain. The Committee was able to make a definite contribution to the problem which it faced, but as will be noted from the list of publications, the subject matter of this early work is still classified.

It was the original intention that, when the war was over, this small committee might develop into an Associate Committee of the usual kind in order to co-ordinate and assist in the development of research studies in Canada related to the physical and mechanical properties of the terrain of this country. The wisdom of this course is now apparent since much of the work which the Committee has been able to do since the end of the war was initiated as a result of the early classified studies. This same pattern was repeated in other cases of special Canadian war research work, demonstrating the flexibility and advantages presented by the operation of Associate Committees of the National Research Council.

Present Status. The Associate Committee on Soil and Snow Mechanics now consists of about twenty Canadian engineers and scientists who hold office each for three years, approximately one-third of the Committee retiring every year so as to ensure reasonable continuity of membership. The Committee normally meets twice a year. At its meetings, it reviews progress in the various branches of its assigned field. It considers applications for research grants in these fields, since it is charged by the National Research with the Administration of a certain amount of money every year for assisting with such work. It seeks to stimulate research work in the four main divisions noted below and to assist with the development of work in these fields throughout the country. It also maintains official liaison with a number of international organizations.

In other countries, soil mechanics might be the only concern of such a committee as the Associate Committee on Soil and Snow Mechanics. The complete surface of Canada, however, is covered with snow for at least a portion of each year so that snow (and correspondingly ice) are matters of equivalent concern in Canada. Correspondingly, large areas of the northern part of Canada are covered by what is called "muskeg" which must be studied if the terrain of Canada is to be properly understood. Finally, much of the ground in the North of the Dominion is found to be perennially frozen. To such soil, the term permafrost has been applied. It introduces many special problems and has already shown itself to be a special field of scientific inquiry.

Georgia Institute of Technology

On February 29, the new Georgia Tech-State Highway Department research building was dedicated. This joint venture will provide facilities for routine testing and development for the State, and laboratories for teaching and research in the fields of soil mechanics, structures, and materials for the School of Civil Engineering of Georgia Tech.

Soil mechanics instruction was formally begun at Georgia Tech in 1947 in response to demand by local engineers and contractors. At the same

time a laboratory was established in the basement of the Civil Engineering building for both graduate and undergraduate instruction. By 1952 the space had been outgrown. With nearly 100 students taking undergraduate instruction each year and 15 taking graduate courses, the 800 square-foot laboratory could not support both teaching and research.

In 1954 the Joint Laboratory Building was announced. This is located on the Georgia Tech. campus but is owned by the Highway Department. The arrangement is unique in that all the facilities will be completely separate. No laboratories or equipment will be shared between the School and the Highway Department. The function of the School will be to provide consultants, and to undertake research for the Highway Department and to aid in the training of their staff.

Soil Mechanics Laboratory: The Soil Engineering Laboratory consists of a main laboratory 42 feet by 50 feet with an adjoining store room and a damp room which together provide about 2300 square feet of space.

In addition to standard identification test apparatus, a stereo-microscope is available for examination of grain size and shape. Equipment for electron micrographs, x-ray diffraction, and differential thermal analysis are available at the Engineering Experiment Station. These are currently employed in the study of kaolin for the mineral producers of the State.

Strength testing equipment includes four 3000-pound platform-scale types for triaxial shear and unconfined compression testing and three controlled-strain machines, two of which are Georgia Tech - designed hydraulic devices employing proving rings for load measurement.

Three triaxial shear chambers are available, one for 2.8 inch-diameter, one for 1.4 inch-diameter, and the third for 1.4 to 2.8 inch-diameter specimens. Two are new Georgia Tech designs employing linear ball bearings for minimum piston friction. The chambers are equipped with pressure regulators to maintain chamber pressures and with devices for volume change measurement and for pore water pressure measurement.

A 2.5-inch diameter direct shear device with a lever system for normal loads and a controlled strain loading is used for shear testing of consolidated soils. The Georgia Tech. design is different in that it employs a pair of stainless steel rings which are clamped together at the center to form a sample cutter. The sharpened edge of one ring cuts a cylindrical sample which is embedded inside the pair of rings. The rings are then clamped into the shear box where they become the sample holder, with one ring shearing across the other. In this way the disturbance produced by transferring the sample from a separate cutter to the shear box is eliminated.

The Georgia Tech consolidometers employ a similar method to reduce sample disturbance. The consolidometer ring is stainless steel about 1/10 inch thick with a sharp cutting edge which trims the soil to a very close fit. The ring then slides into a floating-ring assembly which uses a plexiglas guide tube to center the sample on the lower porous plate. Pyramidal upper porous plates apply the load to the soil through a balljoint. Two sizes are used: 1.4 inch-diameter to study the variations in soil compressibility and 2.8 inch-diameter for orthodox testing. In addition, Casagrande-type fixed ring consolidometers are used for research.

Research: Considerable research is under way, all of which is supported entirely by the interest of the graduate students and the staff. A series of investigations are under way which involve the compaction of cohesive soils and the factors involved in efficient compaction. A second series of projects

involves the lateral stresses produced by the compaction of soils, particularly the residual stresses after the compaction process is complete. A third series of projects is concerned with the bearing capacity of both footing and deep pier foundations, as determined analytically and by full-scale load tests. A fourth series is a continuing study of the physical properties of the decomposed rocks of the Southern Piedmont region. Other research projects under way include a study of grout penetration, the determination of maximum and minimum void ratios, vane shear, and granular stabilization. Instruction: All undergraduate civil engineering and architectural engineering students take a basic course in soil engineering in their senior year. This course, five quarter hours, includes soil physics, basic soil mechanics and applications to simple design problems. A second course is provided as an undergraduate elective. Three basic graduate courses are taught. The first covers the theory and practice of soil stabilization. The second covers advanced foundation theory and design. The third includes seepage and stability problems. Two advanced courses, one in soil physics and the other in applied soil mechanics are available for students specializing in soils. Two degree programs are maintained: the M.S. in Civil Engineering and the Ph.D., both with specialization in soil mechanics.

NINTH CANADIAN SOILS CONFERENCE

In addition to the material he sent us on the Associate Committee on Soil and Snow Mechanics, Mr. Legget was also kind enough to contribute the following account of the Ninth Canadian Soil Mechanics Conference, which was held in December in Vancouver.

"Geologists, agricultural soil scientists and even forestry scientists joined with civil engineers from all parts of Canada, to the number of over 100, at the Ninth Canadian Soil Mechanics Conference to demonstrate the broad interests which now exist in the scientific study of soils. This Conference was held on the beautiful campus of the University of British Columbia in Vancouver, by invitation of the University. It is sponsored each year by the Associate Committee on Soil and Snow Mechanics of the National Research Council as part of its function of stimulating research in the field of soil mechanics throughout Canada and of ensuring that the results of research are put to use in engineering practice.

"These annual conferences are usually held in Ottawa, but every third year the meeting is convened at some other centre. This Ninth conference was the first to be held so far from Ottawa and is believed to be the first such general meeting held for the discussion of soil mechanics on the Pacific Coast of Canada. Because of its locale, the program for the conference differed somewhat from the usual Ottawa meetings, since it provided an opportunity for presenting publicly a general review of soil characteristics and soil problems of British Columbia in general, and of the Lower Mainland in particular.

"The two-day program had been arranged by a local committee, the nucleus of a continuing Pacific Coast soil mechanics group, headed by Mr. Charles F. Ripley, Consulting Engineer of Vancouver. Papers were given on the physiography, climate, and general soil types of British Columbia which illustrated the remarkable variation in physical conditions in this western province of the Dominion. It was pointed out by Dr. W. H. Matthews

(Dept. of Geology, U.B.C.) that the climate of three of the cities of British Columbia, separated by only a few hundred miles, corresponds to the climates of Istanbul, Moscow, and Bergen (Norway). These climatic variations are reflected in the great variety of soil types found in the province.

"These soil types were described from the agricultural point of view and also with reference to the geological origin and transportation. Dr. J.E. Armstrong of the Geological Survey of Canada, gave a summary of his seven years of field work in studying the soils of the Lower Mainland and showed clearly how geological processes can influence the properties of the soils of most concern to civil engineers in their work.

"Other papers described various engineering problems encountered with local soils including serious building settlements on some of the organic silts which are found along the British Columbia coast. Mr. H. Thurber of the B.C. Department of Highways described a serious landslide which had occurred at Golden in connection with the building of the new Trans-Canada Highway.

"The Conference joined with the Vancouver Branch of the Engineering Institute of Canada and the British Columbia Association of Professional Engineers in a joint evening meeting at which illustrated papers on the soil mechanics aspects of the St. Lawrence Seaway project were presented by Messrs. F. L. Peckover and B. J. Bazett of Montreal and Toronto respectively.

"Two guests from the United States received a special welcome at the conference. Dr. D. R. Mullineaux of the Engineering Geology Division of the United States Geological Survey and Mr. W. L. Shannon, a Consulting Engineer from Seattle, who presented a paper on the measurement of earth movement on behalf of Mr. S. D. Wilson, the present chairman of the Soil Mechanics Section of the American Society of Civil Engineers.

"The value of such international contacts was stressed during the business session of the conference which was conducted by Mr. R. F. Legget, Chairman of the Associate Committee. A report was made on Canada's participation in the International Society of Soil Mechanics and Foundation Engineering by Mr. C. B. Crawford, the Canadian secretary for this international body.

"At this meeting there was released for the first time a booklet produced by the Associate Committee, entitled "A Guide to the Field Description of Soils." This publication has been in preparation for several years, indicating the difficulty of producing a compact and yet accurate guide to soil terminology. (Copies are now available at 10 cents each from the Secretary of the Associate Committee, c/o National Research Council, Ottawa.) Announcement was also made of the formal organization of a special Subcommittee on Soil Mechanics under the chairmanship of Mr. Robert Peterson of P.F.R.A., Saskatoon, Saskatchewan.

"Official greetings were brought to the conference at its first luncheon session by Dr. N.A.M. MacKenzie, President of the University of British Columbia, who was supported by Dean Gunning of the Faculty of Applied Sciences and Dean Eagles of the Faculty of Agriculture. At the second luncheon an address was given on engineering geology by Dr. Victor Dolmage of Vancouver. All these speakers stressed the eminent desirability of linking geology and soil science with engineering soil studies, now commonly called soil mechanics, to the mutual benefit of all three disciplines."

FOURTH INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

Plans for the Fourth International Conference on Soil Mechanics and Foundation Engineering, to be held in London, England, August 12 to 24, 1957 (that's a year from next August, gentlemen - put those shirts back in the drawer), have reached the stage at which Bulletin No. 1 is expected at any moment as this is written.

By way of elaboration of the anticipated contents of Bulletin No. 1, some excerpts from an information letter to the Secretary of the U.S. National Council from the International Secretary are reproduced following:

"....it is most desirable that papers to be presented to the Conference should be of the highest standard, and the Organising Committee have therefore evolved a system of presentation which they hope will be agreeable.... and effective in practice. It is requested that the papers be submitted in two stages: (a) Notification, and (b) Final Submission.

(a) Notification

1. Authors who wish to submit papers to the Conference should send a statement not exceeding 300 words, giving an indication of the contents of their proposed papers to the Secretary of the U.S. National Council.
2. The Executive Committee of U.S. National Council should then decide which of the proposed papers should be accepted.
3. The Secretary of U.S. National Council should then forward three copies of the statements relating to the accepted papers to the International Secretary not later than 1 June, 1956.* This will enable the Organising Committee to estimate how many papers are likely to be presented at the Conference.

(b) Final Submission

1. The Secretary U.S. National Council should inform these authors whose proposed papers have been accepted to proceed with the preparation of their papers.
2. The final copies of the papers should be sent to the Secretary of U.S. National Council for approval by the U.S. National Committee.
3. Three copies of each of the papers approved by that Committee should be sent to the International Secretary not later than 1 August, 1956.

The Organising Committee feel that it is very much better for each National Society to decide which papers should be forwarded to the Conference than for this decision to have to be made without full knowledge by the Organising Committee, and they most earnestly request to be sent only those papers which are considered of sufficient merit to justify publication in the Proceedings."

The Secretary of the U.S. National Committee is Mr. John Lowe III, Room #400, 62, West 47th Street, New York 36, New York.

The International Secretary is Mr. A. Banister, the Institution of Civil Engineers, Great George Street, London, S.W.1. England.

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*[Change of date from 1 March approved by International Secretary following request of U.S. National Council. Ed.]

PROMINENT NORWEGIAN SOIL MECHANICS EXPERT VISITING THE UNITED STATES

By the time this Newsletter reaches its readers, Mr. Laurits Bjerrum, one of Norway's leading engineers and research scientists in the field of soil mechanics, will have reached the United States on a visit of about two months' duration.

His itinerary alone marks him as a man of outstanding qualifications, one of which must be an unlimited supply of energy: During the first week in April he is scheduled to visit a number of organizations, institutions, and projects in New York, Washington, and neighboring localities, including a visit to Princeton University. The following week takes him to Vicksburg, from where he plans to go to Mexico City for the third week in April. During the last week in April he plans to be in Denver, after which he has scheduled a week in the Middle West to visit Northwestern University, Purdue University, and the University of Illinois, in addition to a number of major engineering projects of interest to him. The second week of May will find him in Ottawa and environs; and the next week, at Harvard, M.I.T., and in the Boston area.

At this point we assume he will take a well-earned vacation - or at least a couple of aspirin tablets.

Mr. Bjerrum's name has rapidly become familiar in geotechnical circles during the last few years as a result of his brilliant work with the Norwegian Geotechnical Institute in Oslo. His presence in this country will be welcomed by engineers and other scientists in the field of soil mechanics, and the numerous talks that he is scheduled to deliver throughout his tour will undoubtedly attract all who can arrange to attend.

Mrs. Bjerrum is accompanying her husband.

RESEARCH BEQUEST IN ST. LOUIS

From H. N. Reitz comes the announcement that a St. Louis firm has donated the sum of $3000.00 to be administered through the local section, ASCE, for research at Washington University on the general drought problem as it has developed in St. Louis. Mr. Reitz, who is Acting Chairman of the Department of Civil Engineering at Washington University, reports that ideas are in abundant supply and activity is about to begin.

ANNOUNCEMENTS

KNOXVILLE CONVENTION

University of Tennessee June 4 - 8, 1956

Planning of Soils Division activities for this meeting appears to be in an admirably advanced stage, under the direction of E. A. Whitehurst (112 Perkins Hall, University of Tennessee, Knoxville). Two sessions have been arranged, with papers to be presented as follows:

Session I:

"Moisture Conditions Under Flexible Airfield Pavements" by J. F. Redus, Engineer, Flexible Pavement Branch, Waterways Experiment Station, Vicksburg, Mississippi.

"Statistical Analysis of Base Course Specifications" by F. H. Kellogg, Dean of Engineering, University of Mississippi.

"Investigation of the Effect of Repetitive Loading on Compacted Cohesive Soil" by D. R. Freitag, Chief, Soil Stabilization Section, Waterways Experiment Station, Vicksburg, Mississippi.

Session II:

"Building Foundations in the Appalachian Valley Region" by G. F. Sowers, Professor of Civil Engineering, Georgia Institute of Technology.

"Some Factors Influencing the Performance of Earth Dams" by J. L. Sherard, Partner-Woodward, Clyde and Associates, Denver.

"Research on the Consolidation of Clays" by G. A. Leonards, Associate Professor of Soil Mechanics, Purdue University.

R. E. Fadum, Head of the Department of Civil Engineering at North Carolina State College in Raleigh, and S. J. Johnson, of Moran, Proctor, Mueser and Rutledge, are slated to preside.

PITTSBURGH CONVENTION

William Penn Hotel October 15-19, 1956.

Pittsburgh, Pennsylvania.

JULY NEWSLETTER

Deadline date for arrival at this office of contributions for the July Newsletter: May 20, please.

Howard P. Hall, Editor
Department of Civil Engineering
Northwestern University,
Evanston, Illinois.

Session I. [illegible]

"[illegible] Conditions [illegible] Soils," by J. H. [illegible], Engineer, [illegible] Branch, Waterways Experiment Station, Vicksburg, Mississippi.

"[illegible] analysis [illegible]," by [illegible], Dean of Engineering, University of Mississippi.

"Determination of the [illegible] of [illegible] Loading on Compacted [illegible]," by H. B. [illegible] [illegible] Experiment Station, Vicksburg, Mississippi, and [illegible]

Session II. [illegible]

"[illegible] in the Mississippi Valley [illegible]," by [illegible] of Civil Engineering, [illegible]

"Some factors influencing the Performance of [illegible]," by [illegible], Engineers, [illegible]

"[illegible] on the Consolidation of Clays," by [illegible], Associate Professor of Soil Mechanics, [illegible] University.

R. E. Fadum, head of the Department of Civil Engineering at North Carolina State College in Raleigh, and [illegible] Johnson of [illegible] Mechanics and Foundations, are slated to preside.

[illegible]

William [illegible], and [illegible] Garrett Smith, [illegible] North Carolina.

JULY NEWSLETTER

Deadline date for receipt at this office of contributions for the July Newsletter: May 25, please.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

APRIL: 659(ST), 660(ST), 661(ST)^c, 662(ST), 663(ST), 664(ST)^c, 665(HY)^c, 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)^c, 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)^c, 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)^c, 700(PO), 701(ST)^c.

JUNE: 702(HW), 703(HW), 704(HW)^c, 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)^c, 710(CP), 711(CP), 712(CP), 713(CP)^c, 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)^c, 719(HY)^c, 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)^c, 727(WW), 728(IR), 729(IR), 730(SU)^c, 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)^c, 749(SA), 750(SA), 751(SA), 752(SA)^c, 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)^c, 759(SM)^c, 760(WW)^c.

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)^c, 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)^c, 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)^c, 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)^c, 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)^c, 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)^c, 808(IR)^c.

OCTOBER: 809(ST), 810(HW)^c, 811(ST), 812(ST)^c, 813(ST)^c, 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)^c, 820(SA), 821(SA), 822(SA)^c, 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)^c, 832(CP), 833(CP), 834(CP), 835(CP)^c, 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)^c.

DECEMBER: 842(SM), 843(SM)^c, 844(SU), 845(SU)^c, 846(SA), 847(SA), 848(SA)^c, 849(ST)^c, 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)^c, 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)^c, 877 (HW1)^c, 878(ST1)^c.

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)^c, 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1). 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)^c, 903(IR1)^c, 904 (PO1)^c, 905(SA1)^c.

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)^c, 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)^c.

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)^c, 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)^c, 943(EM2), 944(EM2), 945(EM2), 946(EM2)^c, 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)^c, 953(HY2), 954(HY2), 955(HY2)^c, 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

c. Discussion of several papers, grouped by Divisions.

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